PONTIFICIA UNIVERSIDAD CATÓLICA DE CHILE
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COMPARISON OF THE STRUCTURAL DESIGN OF A REINFORCED CONCRETE FRAME BUILDING USING DIFFERENT SPECTRA AND INCORPORATING SEISMIC ISOLATION

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"The drop pierces the rock, not by its

force, but by its persistence."

Ovidio

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ABSTRACT

The increasing frequency and intensity of earthquakes worldwide highlights the urgent need to strengthen seismic safety in building design. In response to this challenge, international cooperation has become essential for developing effective strategies to mitigate seismic risks. The International Platform for Reducing Earthquake Disasters (IPRED) promotes this global collaboration, facilitating the exchange of knowledge and practices in seismic design. This study is part of a collaborative effort initiated by IPRED to compare seismic design approaches across different countries. This research has two main objectives. The first is to compare the structural design of a prototype reinforced concrete residential building using response spectrum analysis with the seismic spectra of Chile and Turkey. The second objective focuses on comparing the structural design of the same building with the incorporation of base isolation, following Chilean standards. The results obtained allow comparisons in aspects such as vibration periods, base shear forces, inter-story drifts, dimensions, and reinforcement of structural elements. The findings reveal significant differences resulting from the application of different seismic spectra to the same building, with the Chilean spectrum generating higher demands overall. Additionally, the implementation of base isolation shows notable advantages by reducing force demands on structural elements and enhancing the overall seismic resilience of the building.

Keywords: Comparative analysis, structural design, response spectrum analysis, base isolation, seismic codes.

RESUMEN

La creciente frecuencia e intensidad de los terremotos a nivel mundial pone de relieve la urgente necesidad de reforzar la seguridad sísmica en el diseño de edificaciones. Ante este desafío, la cooperación internacional se ha vuelto esencial para desarrollar estrategias efectivas que mitiguen los riesgos sísmicos. La Plataforma Internacional para la Reducción de Desastres por Terremotos (IPRED) promueve esta colaboración global, facilitando el intercambio de conocimientos y prácticas en diseño sísmico. Este estudio forma parte de un esfuerzo colaborativo iniciado por IPRED para comparar enfoques de diseño sísmico en diferentes países. El trabajo de investigación tiene dos objetivos principales. El primero consiste en comparar el diseño estructural de un edificio residencial prototipo de hormigón armado mediante un análisis modal espectral, utilizando los espectros sísmicos de Chile y Turquía. El segundo objetivo se centra en comparar el diseño estructural del mismo edificio, incorporando aislamiento basal, bajo la normativa chilena. Los resultados obtenidos permiten realizar comparaciones en aspectos como los períodos de vibración, las fuerzas cortantes basales, las derivas de entre pisos, las dimensiones y los refuerzos de los elementos estructurales. Los hallazgos evidencian diferencias significativas derivadas de la aplicación de distintos espectros sísmicos en una misma edificación, destacándose el espectro chileno por generar una mayor demanda en general. Además, la implementación de aislamiento basal muestra ventajas notables al reducir las cargas sobre los elementos estructurales y mejorar la resiliencia sísmica global del edificio.

Palabras clave: Análisis comparativo, diseño estructural, análisis modal espectral, aislamiento basal, normas sísmicas.

I INTRODUCTION

I.1 Motivation

The International Platform for Reducing Earthquake Disasters (IPRED) promotes global cooperation in seismology to strengthen building codes and reduce earthquake-related risks. In this regard, international cooperation in seismology and seismic engineering is encouraged to improve building code practices worldwide. This intergovernmental scientific platform recognizes the importance of enhancing the safety of buildings and housing as a fundamental and vital priority to reduce risks globally (UNESCO, n.d.).

Improving the safety of buildings and housing worldwide requires strengthening existing alliances and creating new partnerships with a diverse range of stakeholders, from Member Countries to civil society organizations and private institutions. IPRED mobilizes a wide range of partners whose active support and commitment help maximize the link between seismology and seismic engineering. Among the Member States of the platform are Algeria, Chile, Egypt, El Salvador, Indonesia, Japan, Kazakhstan, Mexico, Peru, Romania, and Turkey.

Turkey, through its research center İstanbul Teknik Üniversitesi (İTÜ), organized, as part of the topics addressed at the IPRED-2024 conference, a collaborative study aimed at comparing a prototype residential structure within the framework of local seismic design codes from various countries. In this context, Chile, as an active member through the Pontificia Universidad Católica de Chile (PUC), has agreed to participate in this collaborative study. This document summarizes the work conducted by the author, as a member of the Chilean team from PUC. The document presents the results analysis and design of the case study building when applying the Chilean seismic code. The Turkish and Chilean design spectra are used to assess the building's response in accordance with the Chilean seismic code.

Additionally, as a second phase of this study, the results of implementing seismic isolation in the Chilean model are presented. In order to compare its response and contribute to the objectives of this work.

I.2 Objectives

The objectives of this research are:

1. Describe the Chilean and Turkish seismic design codes.
2. Define the case study building.
3. Compare the structural design of the case study building using response spectrum analysis with the Chilean and Turkish spectra.
4. Compare the structural design of the case study building when incorporating base isolation.

I.3 Organization of the Document

This research work is organized into six chapters. This first chapter includes the motivation for the study, the objectives, and the organization of the document. The second chapter discusses the seismic design codes that will be used to establish the theoretical foundations necessary for this study. The third chapter pertains to the definition of the case study, describing the building's geometry, material properties, loads, masses and seismic design considerations to be applied. The fourth chapter focuses on the structural design of the case study building using the Chilean and Turkish design spectra, followed by a comparative analysis of the results obtained.

The fifth chapter presents the structural design of the case study building with the implementation of base isolation, followed by a comparative analysis with the conventional structure, all based on the Chilean code. Finally, the sixth chapter presents the conclusions of the study, along with potential recommendations for future research in this field.

II SEISMIC DESIGN CODES

This chapter outlines the seismic design codes that form the basis of this research. It provides a detailed summary of the seismic design philosophy and practices applied in Chile and Turkey, aiming to establish the essential theoretical foundations for the case study presented in Chapter 3 of this document.

II.1 Chilean Seismic Design Code (NCh433 and DS61)

II.1.1 Introduction

Chile is one of the most seismically active countries in the world due to the subduction of the Nazca Plate beneath the South American Plate. On average, a destructive earthquake with a magnitude greater than 8.0 occurs every 10 years. This seismic activity has driven the development and periodic updating of seismic design codes, such as the NCh433 (2009), which was revised following the 2010 Maule earthquake with the publication of the Supreme Decree DS 61 (2011).

II.1.2 Basic Principles and Assumptions

The NCh433 code aims to ensure structures that:

1. Withstand moderate-intensity seismic motions without damage.
2. Limit damage to non-structural elements during medium-intensity earthquakes.
3. Prevent collapse during exceptionally severe earthquakes, even if structural damage occurs.

II.1.3 Seismic Zoning and Soil Classification

Chile is divided into three seismic zones (refer to Figures 4.1 a), 4.1 b), and 4.1 c) of the code for detailed reference). Soil classification is based on the shear wave velocity in the upper 30 meters (V_{s30}). Six soil types are recognized (A, B, C, D, E, and F), with specific criteria for classification supported by measurements such as the Standard Penetration Test index (N_1), undrained shear strength (S_u), and other parameters. Detailed descriptions of this classification can be found in DS 61 (2011).

II.1.4 Occupancy Category of Buildings and Other Structures

The NCh433 (2009) classifies buildings and other structures into four occupancy categories (I, II, III, and IV), ranging from least to greatest importance. Table 4.1 of the standard provides detailed information on each category and the types of structures included in each.

II.1.5 Coordination with Other Analysis and Design Codes

The NCh433 (2009) is applied in conjunction with other Chilean codes, the most relevant for this study being: NCh3171 Load combinations (2010), NCh1537 Load analysis (2009), and NCh430 Reinforced concrete structures (2008).

II.1.6 Structural Systems

The following types of structural systems are distinguished in the NCh433 (2009), among others:

- Wall systems and other braced systems: Primarily resist gravitational and seismic forces through axial stress.
- Frame systems: Resist gravitational forces and seismic actions in both analysis directions through frames.
- Mixed systems: Combine walls and frames.

II.1.7 Estimation of Structure Weight

The NCh433 (2009) specifies that for mass calculations, permanent loads must be considered along with a percentage of live loads. This percentage must not be less than 25% for buildings intended for private housing or public use where crowding of people or objects is uncommon, and not less than 50% for structures where such crowding is common.

II.1.8 Seismic Analysis Procedures

The Chilean code establishes two types of seismic analysis:

- Static analysis method
- Response spectrum analysis method

II.1.9 Response Modification Factor

This factor reflects the energy absorption and dissipation characteristics of the resisting structure, as well as the experience with the seismic behavior of various types of structural systems and materials used. Table 5.1 of the NCh433 (2009) establishes the maximum values for response modification factors. For a reinforced concrete frame system, the values are 7 and 11 for R and R_0, respectively. The value of R is used for the static analysis method and the value of R_0, is used to compute R^*, for the response spectrum analysis.

II.1.10 Seismic Deformations

Regarding seismic deformations, the NCh433 (2009) code specifies that horizontal displacements and rotations of story diaphragms must be calculated using reduced forces, including the effect of accidental torsion, considering the following maximum values:

- The maximum relative displacement between two consecutive stories, measured at the center of mass in each of the analysis directions, must not exceed the interstory height multiplied by 0.002.
- The maximum relative displacement between two consecutive stories, measured at any point of the story in each analysis direction, must not exceed the corresponding relative displacement measured at the center of mass by more than 0.001h, where h is the interstory height.

II.1.11 Static Analysis

The Chilean code specifies that the static analysis method can only be used for the seismic analysis of the following resistant structures:

- Categories I and II in seismic zone 1.
- Structures up to 5 stories or 20 meters in height.
- Structures with 6-15 stories if they meet stiffness and base shear conditions.

a) Base Shear

The base shear force is given by:

$$Q_0 = CIP \tag{II.1}$$

Where:

C: Seismic coefficient.

I: Coefficient relative to the building's category (refer to Table 6.1 of NCh433).

P: Total weight of the building at the base level.

b) Seismic Coefficient

The seismic coefficient C is obtained from the expression:

$$C = \frac{2.75\, S\, A_0}{g\, R}\left(\frac{T'}{T^*}\right)^n \tag{II.2}$$

Where:

n, T', S: Parameters related to soil type, with values specified in Table 6.3 of NCh433 (2009).

A_0: Maximum effective acceleration relative to the seismic zone, with values specified in Table 6.2 of NCh433 (2009).

R: Reduction factor according to Table 5.1 of the code.

T*: Period of the mode with the highest translational mass in the direction of analysis.

The C value is limited by the minimum value of:

$$C_{min} = \frac{A_0\, S}{6\, g} \tag{II.3}$$

The maximum value of C is also limited and depends on the reduction factor R. The maximum value is indicated in Table 6.4 of NCh433 (2009).

II.1.12 Response Spectrum Analysis

This method can be applied to structures that exhibit classical normal vibration modes, with modal damping of approximately 5% of the critical damping. The analysis must include all normal modes, ordered by increasing values of natural frequencies, that are necessary for the sum of the equivalent masses for each of the two seismic actions to be greater than or equal to 90% of the total mass.

a) Analysis for Accidental Torsion

The effect of accidental torsion must be considered in one of the following two alternative forms:

- Transverse displacements of the centers of mass (±0.05 times the building width).
- Application of static torsion moments based on accidental eccentricities.

b) Design Spectrum

The pseudo-acceleration design spectrum that determines the seismic resistance of the structure is defined by:

$$S_a = \frac{SA_0\alpha}{R^*/I} \tag{II.4}$$

Where:

α: Amplification factor, which is determined for each vibration mode n, according to the following expression:

$$\alpha = \frac{1 + 4{,}5\left(\frac{T_n}{T_0}\right)^p}{1 + \left(\frac{T_n}{T_0}\right)^3} \qquad \text{(II.5)}$$

Where:

T_n: Vibration period of mode n.

T_0, p: Parameters related to the foundation soil type. See Table 6.3 of NCh433 (2009).

The reduction factor R^* is determined by:

$$R^* = 1 + \frac{T^*}{0{,}10T_0 + \frac{T^*}{R_0}} \qquad \text{(II.6)}$$

Where:

R_0: Value for the structure established according to the provisions of the maximum response modification factors (see Table 5.1 of NCh433).

Finally, the reduction factor R* needs to be adjusted if the resulting base shear is below the minimum or above the maximum base shear specified by NCh433. Below, in Figure II-1, the elastic spectra from NCh433 (2009) for different soil types are shown, considering a residential occupancy category (I=1.0) and a location in seismic zone 3 (A_0=0.40g).

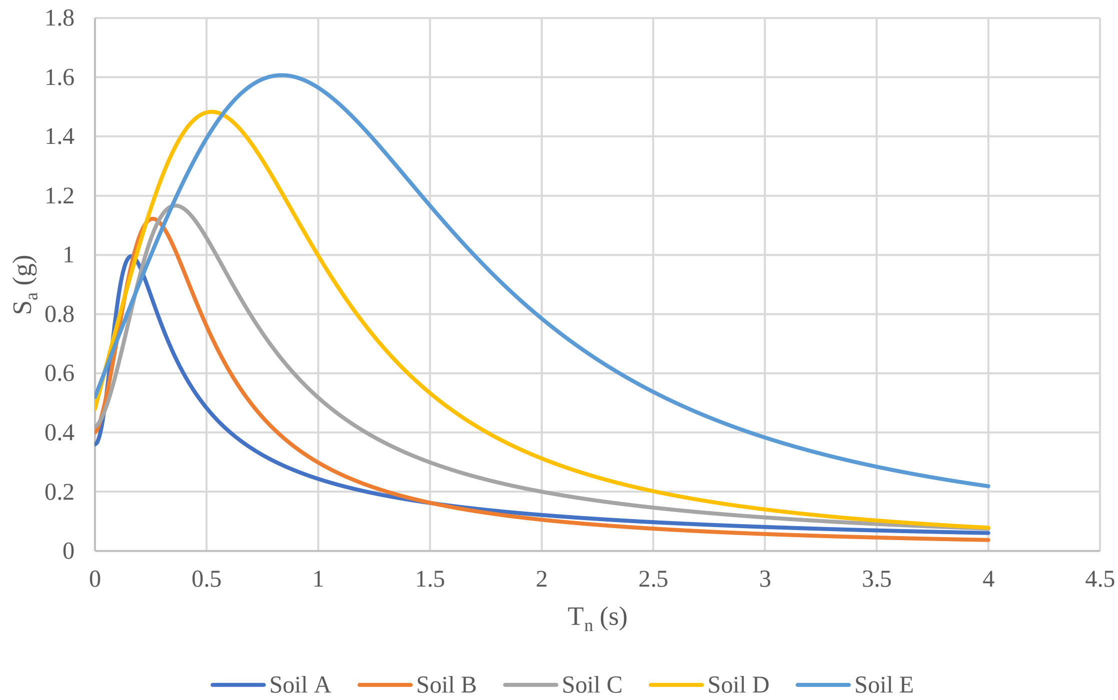


Figure II-1: Elastic pseudo-acceleration spectra according to NCh433 (2009) for different soil types, considering I=1.0 and A_0=0.40g.

II.2 Turkish Seismic Design Code (TBDY-2018)

II.2.1 Introduction

Since the 1940s, Turkey has maintained a regularly updated Seismic Code, culminating in the 2018 Code (TBDY, 2018), which incorporates innovations such as specific seismic movement definitions, design provisions for high-rise buildings and base isolation, deep foundations, mandatory nonlinear analysis in certain cases, and performance evaluation in non-standard practices. The TBDY (2018) includes seismic hazard maps, force- and deformation-based design, seismic evaluation of existing structures, and a design supervision system. Published by the Disaster and Emergency Management Authority (AFAD), it came into effect on January 1, 2019, with implementation overseen by the Ministry of Environment and Urbanization.

II.2.2 Seismic Hazard Map

According to Sucuoglu (2018), the current Seismic Hazard Map of Turkey is not a seismic zoning map but rather a contour map based on geographic coordinates. Seismic hazard is expressed in terms of spectral acceleration rather than PGA. Site-specific spectral acceleration maps for stiff soil sites have been developed for T = 0.2 s and T = 1 s, with return periods of 2475, 475, 72, and 43 years. Additionally, a PGA contour map has been created. All maps are publicly accessible through the official AFAD website.

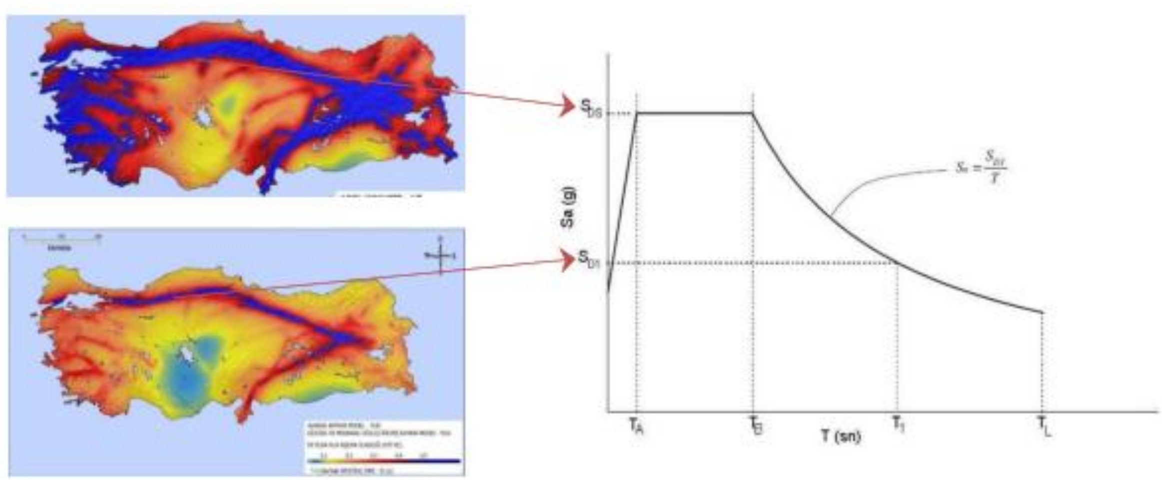
II.2.3 Earthquake Ground Motion Levels

The TBDY (2018) specifies four different levels of earthquake ground motion levels, each associated with different performance objectives. The seismic design levels are classified based on the probability of being exceeded within a given period and their corresponding return periods: DD-1, with a 2% probability in 50 years (2475 years return period), represents the maximum expected seismic motion level; DD-2, with a 10% probability in 50 years (475 years return period), is the standard design level; DD-3, with a 50% probability in 50 years (72 years return period), corresponds to the frequently expected level; and DD-

4, with a 50% probability in 30 years (43 years return period), describes the seismic service level.

II.2.4 Design Spectrum

According to Sucuoglu (2018), spectral acceleration values S_S and S_1 at $T = 0.2\ s$ and $T = 1.0\ s$, respectively are obtained from the associated hazard maps prepared for reference stiff soil sites. Then they are modified with respect to the soil conditions at the project site in order to obtain the design spectral accelerations S_{DS} and S_{D1}. Finally, the design spectrum is constructed as illustrated in Figure II-2. The corner periods T_A and T_B are obtained from the associated ratios of S_{DS} and S_{D1}.



Contour maps for T= 0.2 s and 1.0 s (S_s and S_1, 475 years)

475-year design spectrum (5% damping)

Figure II-2: Construction of the 475-year design spectrum from the 2018 Turkish Seismic Hazard Map (Sucuoglu, 2018).

II.2.5 General Rules for Seismic Design of Building Structures

TBDY (2018) classifies buildings based on their use (BKS=1: critical facilities, BKS=2: high temporary occupancy, BKS=3: others), seismic design based on spectral acceleration, and height to determine the appropriate design procedure. Performance levels include Continuous Operation (CO), Limited Damage (LD), Controlled Damage (CD), and Collapse Prevention (CP). Seismic design combines performance objectives and seismic motions, employing either force-based design (FBD) or performance-based design (PBD). For non-tall buildings, the standard objective is "Controlled Damage" under DD-2 (475 years), while critical buildings have advanced requirements. Tall buildings must remain elastic under DD-3 (43 years) and prevent collapse under DD-1 (2475 years), requiring nonlinear analyses. For existing buildings, a displacement-based elastic analysis is applied.

II.2.6 Force-Based Design of Buildings

Force-based design (FBD) applies to new buildings, except for evaluating tall buildings for collapse prevention and seismically isolated structures. It follows the principles of the 2007 Seismic Code, with improvements in force reduction (R) and overstrength (D) factors, where $R = R\mu * D$. Overstrength arises from factors such as minimum section dimensions and material strength, being crucial for brittle, force-controlled members. Additionally, using effective stiffness in linear elastic analysis allows for a more realistic distribution of forces and deformations, optimizing structural design.

II.2.7 Limitation of Interstory Drift

Limiting interstory drift is necessary to protect fragile non-structural components from lateral deformations imposed by the structural frame. Cracks or damage in non-structural components, particularly masonry infills, significantly reduce the apparent performance of the entire building, even if no damage occurs in the ductile frame members. A flexible separation between the infill and the frame can prevent such damage. The development of these interface connections is encouraged in TBDY (2018), imposing higher drift limits for flexible infill-frame connections and lower limits for direct-contact connections. The TBDY (2018) approach is illustrated in Figure II-3.

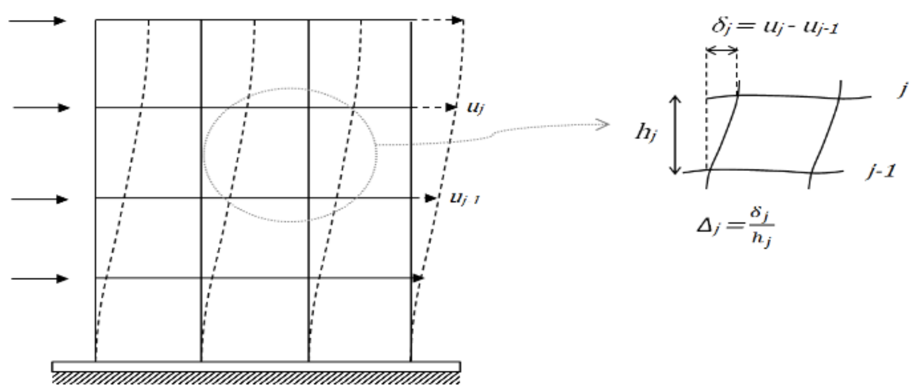


Figure II-3: Interstory drift in a frame and effective interstory drift δ_i, at the i'th story (Sucuoglu, 2018).

The interstory drift limits are as follows:

- Infills rigidly connected to the frame:

$$\lambda \frac{\delta_{i,max}^{(X)}}{h_i} \leq 0.008\kappa \quad \text{(II.7)}$$

- Infills with flexible connections to the frame:

$$\lambda \frac{\delta_{i,max}^{(X)}}{h_i} \leq 0.016\kappa \tag{II.8}$$

Here, λ is the spectral acceleration ratio of DD-3 to DD-2, typically ranging between 0.4 and 0.5. κ is 1.0 for concrete buildings and 0.5 for steel buildings. Notably, in flexible, long-period frames, the interstory drift limit may govern the design more than the design forces.

II.3 Chilean Seismic Isolation Design Code (NCh2745:2013)

II.3.1 Introduction

The NCh2745 (2013) code sets the requirements for the seismic design and analysis of buildings with isolation systems. Based on international experiences and adapted to the seismic reality of Chile, it aims to protect both life and structural and non-structural integrity during severe earthquakes. This code has been harmonized, as far as possible, with the Chilean seismic design code for buildings, NCh433.

II.3.2 Fundamental Principles of Seismic Isolation

Seismic isolation horizontally decouples the structure from ground motion, concentrating deformations in specialized devices (seismic isolators). These systems increase the horizontal flexibility of the structure, lengthening its natural period and reducing the transmission of seismic energy to the superstructure. The basic principles are:

- Flexibilization: Introducing a "soft story" to reduce transmitted forces.
- Increased damping: Reducing deformation demands and shear forces.

The most commonly used systems include:

- Low Damping Rubber (LDR) and High Damping Rubber (HDR) isolators.
- Lead-Rubber Bearing (LRB) isolators.
- Friction Pendulum Systems (FPS)

II.3.3 Design Philosophy

The NCh2745 (2013) defines two main seismic levels:

- Design Earthquake (SDI): With 10% probability of exceedance in 50 years.
- Maximum Possible Earthquake (SMP): With 10% probability of exceedance in 100 years.

The design must ensure that the structure can withstand minor and moderate earthquakes without damage, while also supporting severe earthquakes without failure of the isolation system or significant damage.

II.3.4 Selection Criteria

The design of structures with seismic isolation must consider factors such as location, site characteristics, vertical acceleration, properties of cracked sections of concrete and masonry elements, the building's purpose, configuration, structural system, and height (NCh2745, 2013).

For buildings with seismic isolation, the importance factor *I* should always be taken as 1.0, regardless of the destination category. The seismic zoning follows the provisions of NCh433 (2009), and the soil classification is grouped into the following equivalent categories based on their typology: I (A), II (B), III (C and D), and IV (E and F).

The seismic isolation systems considered adequate must meet the following requirements:

- Maintain stability for the required design displacement.
- Provide resistance that does not decrease with increased displacement.
- Prevent degradation of stiffness and resistance under cyclic loads.
- Have a well-defined and repeatable force-deformation constitutive relationship.

II.3.5 Structural Model

The code allows the use of both linear and nonlinear models, as well as static and dynamic analysis procedures. The main considerations are:

- Isolation system model: Representing three-dimensional effects, load distribution, and property variability.

- Equivalent linear model: Applicable to elastomeric bearings, with additional verification for frictional systems.
- Nonlinear model: Required to assess the constitutive behavior of devices with deformation velocity dependence.
- Superstructure model: Should be modeled with a level of detail similar to that of a conventional building. However, the uncertainty in the superstructure model's response is reduced due to the isolation system.

II.3.6 Static Analysis

Used when specific conditions of location, soil type, and structural configuration are met. It requires the isolation system to:

- Be capable of achieving design and maximum possible displacements.
- Maintain force-deformation properties independent of velocity and vertical loads.

a) Minimum Lateral Displacements

The isolation system must be designed and constructed to support, at a minimum, lateral seismic displacements acting in the direction of the two principal axes of the structure as follows:

$$D_D = \frac{C_D}{B_D} \qquad \text{(II.9)}$$

Where:

$$C_D = \begin{cases} 200\ Z\ [mm], for\ Soil\ I \\ 300\ Z\ [mm], for\ Soil\ II \\ 330\ Z\ [mm], for\ Soil\ III \end{cases} \quad \text{(II.10)}$$

B_D corresponds to the damping response modification factor and is obtained from Table 2 of the NCh2745 (2013). Z is a factor that depends on the seismic zoning (see Table 5 of the NCh2745)

b) Effective Period Corresponding to the Design Displacement

The effective periods of the isolated structure corresponding to the design displacement T_D and the maximum displacement T_M should be determined using the force-deformation characteristics of the isolation system according to the formulas:

$$T_D = 2\pi \sqrt{\frac{W}{k_{D,min} g}} \quad \text{(II.11)}$$

$$T_M = 2\pi \sqrt{\frac{W}{k_{M,min} g}} \quad \text{(II.12)}$$

c) Maximum Displacement

The maximum displacement of the isolation system, D_M, in the most critical horizontal direction, should be calculated using the formula:

$$D_M = \frac{C_M}{B_M} \quad (\text{II.13})$$

Where:

$$C_M = \begin{cases} 200\ M_M\ Z\ [mm], for\ Soil\ I \\ 300\ M_M\ Z\ [mm], for\ Soil\ II \\ 330\ M_M\ Z\ [mm], for\ Soil\ III \end{cases} \quad (\text{II.14})$$

M_M is obtained from Table 3, B_M is obtained from Table 2, and Z from Table 5 of the NCh2745 (2013).

d) Minimum Lateral Forces

The isolation system, foundation, and all structural elements beneath the isolation system should be designed and constructed to resist a minimum lateral seismic force, V_b, using all appropriate capacity, deformation, and resistance requirements for non-isolated structures:

$$V_b = \frac{k_{D,máx} D_D}{R_b} \quad (\text{II.15})$$

Where R_b must not exceed 1.5 for the foundation and all structural elements under the isolation system and must be equal to 1.0 for the isolation system.

The structure above the isolation system should be designed to resist at least a shear force, V_s, using all appropriate capacity, deformation, and resistance requirements for non-isolated structures:

$$V_s = \frac{k_{D,máx} D_D}{R_s} \tag{II.16}$$

The reduction factor R_s is based on the type of lateral load-resistant system used in the superstructure (as per Table 4 of the NCh2745).

Limits for V_b and V_s:

- The value of V_b at the isolation interface should not be less than V_s.
- The value of V_s at the isolation interface should not be less than:
    - The lateral seismic force required by NCh433 for a fixed-base structure of the same weight W and a period equal to the isolated structure's T_D.
    - The minimum shear force required by NCh433, considering I=1.0 and S=1.0 for all soil types.
    - The shear force corresponding to the design wind load.
    - The seismic lateral force required to fully activate the isolation system, multiplied by 1.5 (equivalent to the system's yield level or static friction in sliding systems).
- V_s should not exceed the value determined by the elastic spectrum

e) Limit of Interstory Displacement

For the stories of the superstructure, the maximum relative displacement between two consecutive stories, measured at the center of mass in each direction of analysis, should not exceed the height of the story multiplied by 0.002.

II.3.7 Dynamic Analysis

Dynamic analysis is performed using two methods: Response spectrum and Response history analysis. Response spectrum analysis is applicable when the structure is located on soil types I, II, or III, and the isolation system meets the requirements specified in the standard. Response history analysis must be used when the criteria for response spectrum are not met, and it is applicable to any structure with seismic isolation. Additionally, a specific design spectrum should be considered when the structure is located on soil type IV, near an active fault, or has an oscillation period greater than 3.5 seconds.

a) Isolation System and Substructure Elements

Equations II-17 and II-18 correspond to modifications of Equations II-9 and II-13, aiming to include the influence of the superstructure's flexibility:

$$D_D' = \frac{D_D}{\sqrt{1 + \left(\frac{T}{T_D}\right)^2}} \qquad \text{(II.17)}$$

$$D_M{}' = \frac{D_M}{\sqrt{1 + \left(\frac{T}{T_M}\right)^2}} \qquad (\text{ II.18 })$$

Where:

T: Period of the superstructure with a fixed base and elastic behavior.

b) Structural Elements of the Superstructure

The design shear in the superstructure must be at least 80% of V_s for regular configurations, or 60% if a response history analysis is performed. For irregular configurations, the shear should not be less than V_s, but it may be reduced to 80% if response history analysis is applied. Additionally, the specified limits for V_b and V_s must be met.

c) Design Spectra

The design spectrum, according to NCh2745 (2013), is defined by the Newmark and Hall spectrum, as shown in Figure II-4, and is associated with soils of types I, II, and III.

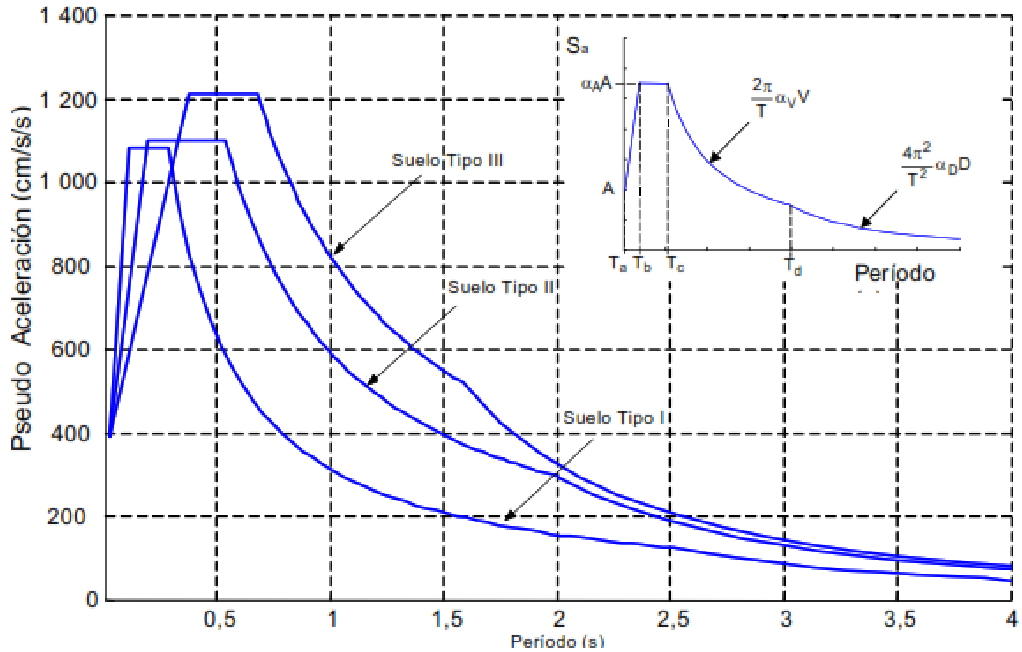


Figure II-4: Pseudo-acceleration design spectrum according to NCh2745:2013 for Soil Types I, II, and III (NCh2745, 2013).

For structures with an isolated period greater than 3.5 s, located on type IV soils, or within 10 km of an active fault, a site-specific spectrum is required. In other cases, the design spectrum should be used, scaled by the Z factor from Table 5 and with the values from Table 6 of NCh2745 (2013). For isolation systems with friction pendulums, the vertical component of the earthquake is also considered, defined as 2/3 of the horizontal spectrum. The design spectrum must be calculated for the design earthquake, without being lower than the minimum spectrum of this standard, and for the maximum possible earthquake, amplified by the M_M factor. In both cases, if a site-specific spectrum is calculated, it should not be less than 80% of the general spectrum defined by the standard. This spectrum must be used to determine the maximum total displacement and forces for the design and testing of isolation systems, ensuring that the minimum requirements are met to guarantee the seismic safety of the structure.

d) Forces and Displacements in the Isolated Structure

For the response spectrum analysis of an isolated structure, the nonlinear force-deformation characteristics of the isolators must be represented using equivalent linear properties, calibrated to achieve the same cyclic energy dissipation as that obtained from their actual behavior. This analysis is iterative due to the dependence of secant properties on angular deformation. The design forces and displacements in the key elements of the lateral force-resisting system can be calculated with a linear elastic model, as long as the equivalent elastic properties are based on the maximum effective stiffness of the isolation system and all system elements are linear.

e) Response Spectrum Analysis

The response spectrum analysis assumes that the modal damping ratio in the fundamental modes of the isolated structure, determined by the characteristics of the isolation interface, is higher than in the modes of the superstructure. For the fundamental modes, the design spectrum is divided by the β_D factor, and for the other modes, β_D values related to the damping of the superstructure fixed to the ground are used. This analysis assumes classical damping and uses two modal damping values. The β_D factor for the fundamental modes is taken as the lesser of the effective damping of the isolation system and $\beta = 0{,}30$.

f) Interstory Displacement Limits

The maximum interstory displacement corresponding to the design lateral force (calculated considering the *R* actor of the superstructure or substructure as applicable), including the horizontal displacement due to the vertical deformation of the isolation system, must not exceed the following limits:

- The maximum ratio of the structure's interstory displacement to the story height above and below the isolation system, calculated using response spectrum analysis, must not exceed 0.0025.
- The maximum ratio of the structure's interstory displacement to the story height above and below the isolation system, calculated by response history analysis considering the force-deformation characteristics of the nonlinear elements of the lateral force-resisting system, must not exceed 0.003.

III CASE STUDY BUILDING

III.1 Geometry

The case study building is a five-story, three-dimensional, reinforced concrete moment-resisting frame, designed in accordance with local seismic codes. Figure III-1 presents its geometry through perspective and plan views. The main characteristics of the case study building are as follows:

- Number of stories: 5
- Story height: 3 m
- Total height: 15 m
- Plan dimensions: 4 x 3 bays (20 m x 13.5 m)

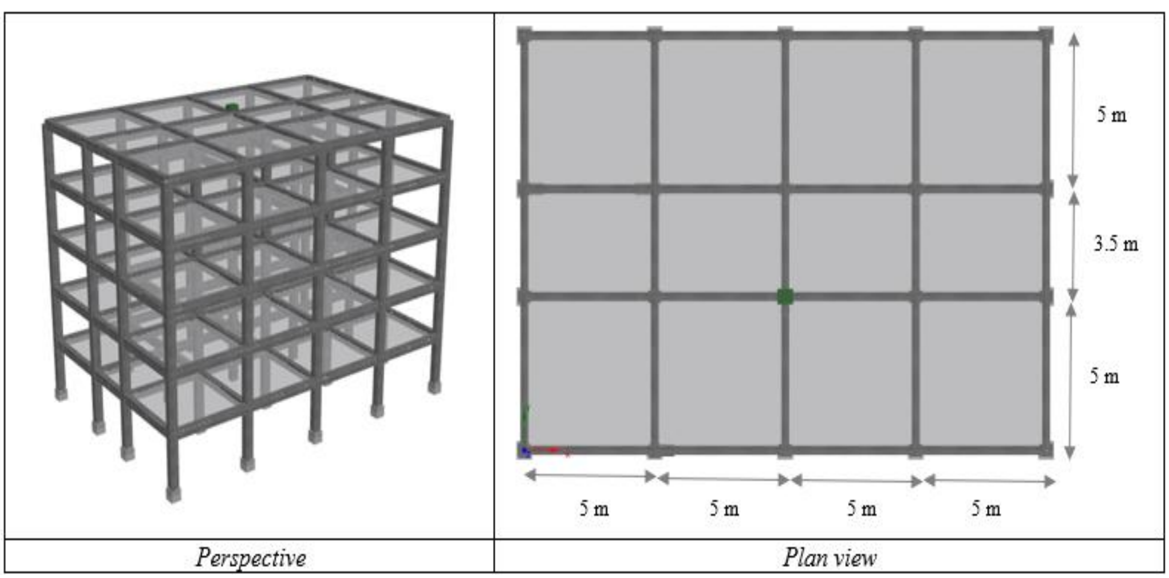


Figure III-1: Three-dimensional and plan views of the case study building

The structure is a residential-type building, regular in plan, and does not consider architectural aspects. The proposed dimensions were the result of discussions among the representatives of several IPRED member states (Chile, Turkey and Japan), who agreed that would be a typical and common geometry for buildings with this function, which is why it was chosen as the case study.

III.2 Material Properties

The structure is defined as a reinforced concrete building, and the following material properties were agreed upon:

- **Concrete:**
  - Type: G25
  - Compressive strength: 25 MPa
  - Self-weight: 24.51 kN/m³
  - Modulus of elasticity (E): 23500 MPa

- **Reinforcing Steel:**
  - Type: A630-420H
  - Yield strength: 420 MPa
  - Ultimate strength: 630 MPa
  - Self-weight: 76.98 KN/m³
  - Modulus of elasticity (E): 200.000 MPa

III.3 Loads and Masses

III.3.1 Load Definition

For the definition of loads and mass assumptions, the team from Turkey provided data on the self-weight of structural elements (columns, beams, and slabs) and perimeter walls, expressed as total values per story. Similarly, the imposed loads assigned to the slabs were determined using a representative value per story, divided between the live load, in accordance with the defined functionality, and a load corresponding to partition walls. The structural mass is considered as 100% of the dead load plus 30% of the live load, following the requirements of Turkish regulations. Table III-1 summarizes the dead loads, live loads, and masses per story as originally provided by the Turkish team.

Table III-1: Dead loads, live loads, and story masses defined by the Turkish team.

Story #	Dead Load (KN)					Live Load (KN)		Story Masses (tons)
	Columns	Beams	Slab	Finishing	Perimeter Wall	Slab	Infill Wall	DL+0.3L
1	240	502.4	809	404.8	450	540	400	274
2	240	502.4	809	404.8	450	540	400	274
3	240	502.4	809	404.8	450	540	400	274
4	240	502.4	809	404.8	450	540	400	274
5	120	502.4	809	0	0	540	0	162.4

In Table III-1, it can be observed that the dead loads include the self-weight of the structural elements, which implies that the initial sections of the building have already been considered. To determine these sections, a detailed quantity computation was performed for each element, yielding the following values:

- Columns: 60 m
- Beams: 147.50 m
- Slabs: 270 m²
- Perimeter walls: 67 m

Based on this data, the dimensions of the structural elements were estimated, as shown in Table III-2.

Table III-2: Dimensions of the structural elements for the case study structure, according to the data provided by the Turkish team.

Structural Element	**Stories 1 to 4**			**Story 5**		
	Thickness (m)	**Height (m)**	**Width (m)**	**Thickness (m)**	**Height (m)**	**Width (m)**
Slabs	0.12	-	-	0.12	-	-
Beams	-	0.30	0.50	-	0.30	0.50
Columns	-	0.40	0.40	-	0.30	0.30

The dead loads were calculated excluding the self-weight of the structural elements. For the live load, it was verified that the provided value of 2 kPa was consistent with the values established in Table 4 of the Chilean standard for permanent and usage loads (NCh1537, 2009) for residential buildings. Table III-3 presents a summary of the dead and live loads applied per square meter and linear meter in all models of this study.

Table III-3: Dead and live loads for the case study structure.

Story #	**Dead Load (D)**			**Live Load (L)**	
	Finishing (KPa)	**Infill Wall (KPa)**	**Perimeter Wall (KN/m)**	**Live Load (L) (KPa)**	**Roof Live Load (L_r) (KPa)**
1	1.50	1.48	6.72	2.00	0.00
2	1.50	1.48	6.72	2.00	0.00
3	1.50	1.48	6.72	2.00	0.00
4	1.50	1.48	6.72	2.00	0.00
5	0.00	0.00	0.00	0.00	2.00

III.3.2 Load Combinations

The load combinations used correspond to those defined in section 9.1.1 of the Chilean standard for general provisions and load combinations, NCh3171 (2010):

- U1 = 1.4D
- U2 = 1.2D + 1.6L + 0.5Lr
- U3 = 1.2D + 1.6Lr + L
- U4 = 1.2D + 1.4Ex + L
- U5 = 1.2D + 1.4Ey + L
- U6 = 0.9D + 1.4Ex
- U7 = 0.9D + 1.4Ey

Where:

D: Dead load.

L: Live load.

Lr: Roof live load.

Ex, Ey: Seismic loads in the X and Y directions, respectively.

III.3.3 Structure Weight

The case study structure must be analyzed following the recommendations of NCh433 (2009). Therefore, for the mass calculation, 100% of the permanent loads plus a percentage of the live load, which must not be less than 25% (D + 0.25L), should be considered.

III.4 Model

III.4.1 General Considerations

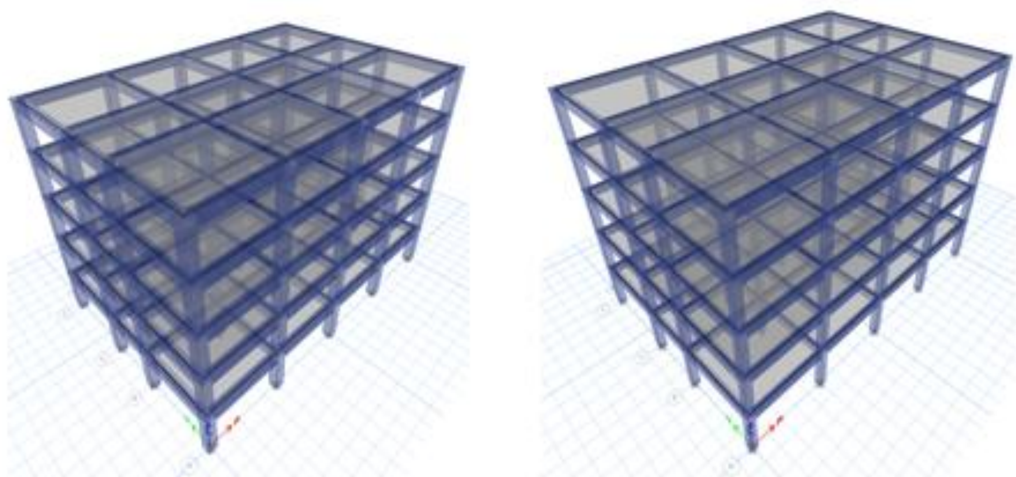
The structural models were developed using the structural design software ETABS v20.3.0, based on the previously defined geometry, loads, and masses. The following general considerations were applied in the modeling process:

- Material Properties: The previously described properties for G25 concrete and A630-420H reinforcing steel were used.
- Slab Modeling: Slabs were modeled as Shell elements to properly represent their bending and torsional behavior.
- Beams and Columns: Beams and columns, with rectangular sections, were represented using Frame elements.

- Column Boundary Conditions: First-story columns were fully fixed at their base, applying displacement and rotational restraints at the bottom node in all directions.
- Load Distribution:
  - The dead load of the slabs, including finishes, fill loads, and live loads acting on the slabs, was modeled as uniformly distributed loads per unit area.
  - The loads from perimeter walls were assigned to the perimeter beams as distributed loads per linear meter.
- Slab Behavior: Slabs were assumed to act as rigid diaphragms, ensuring an adequate distribution of inertial forces at all levels. A 5% eccentricity was considered in all story diaphragms.
- Beam-Column Joints: The connections between beams and columns were considered fully rigid, preventing relative rotations between the connected elements.
- Response Spectra: Different elastic response spectra were introduced depending on the type of structure (conventional or isolated).
- Response Spectrum Analysis: For vibration analysis, classical vibration modes were adopted with a modal damping ratio of 5% relative to the critical damping, following standard practices for seismic assessment.

III.4.2 Conventional Structure

The conventional structure was modeled considering two types of structures, each incorporating a different response spectrum (Chilean or Turkish) within the software. Except for this distinction, both models share the same general modeling conditions. Figure III-2 presents the corresponding models for each case.



(a) (b)

Figure III-2: Three-dimensional view of the conventional structure models for the Chilean (a) and Turkish (b) response spectra.

III.4.3 Isolated Structure

The isolated structure model is based on the same characteristics as the conventional structure, with the following modifications:

- Isolation Level: A new isolation level is introduced at the base of the structure, consisting of beams and slabs.
- Isolators:
  - Modeled as Link elements with a fixed base.
  - The "Rubber Isolator" Link type is used, where:

- The vertical direction (U1) is fixed to restrict displacement.
- In the horizontal directions (U2 and U3), the calculated effective stiffness is introduced.
- The effective damping is set to zero, as its effect is incorporated into the response spectrum.
- The shear deformation location (inflection point) is assumed to be at half the height of the selected isolator (0.25 m).

- Response Spectrum:
  - A reduction of the spectrum is applied using the effective damping factors (B_D or B_M), which introduces a discontinuity in the displacement amplification region.
  - This discontinuity reflects the additional damping effect provided by the isolators, as specified in NCh2745 (2013).
  - When entering the response spectrum into the software, the classical damping of 0.05 is maintained.
- Scaling Factors:
  - Different scaling factors are considered for the Ex and Ey load cases, depending on the analyzed part of the structure:
  - Superstructure: $1/R_s$
  - Substructure: 1.0

Figure III-3 presents the three-dimensional model of the isolated structure.

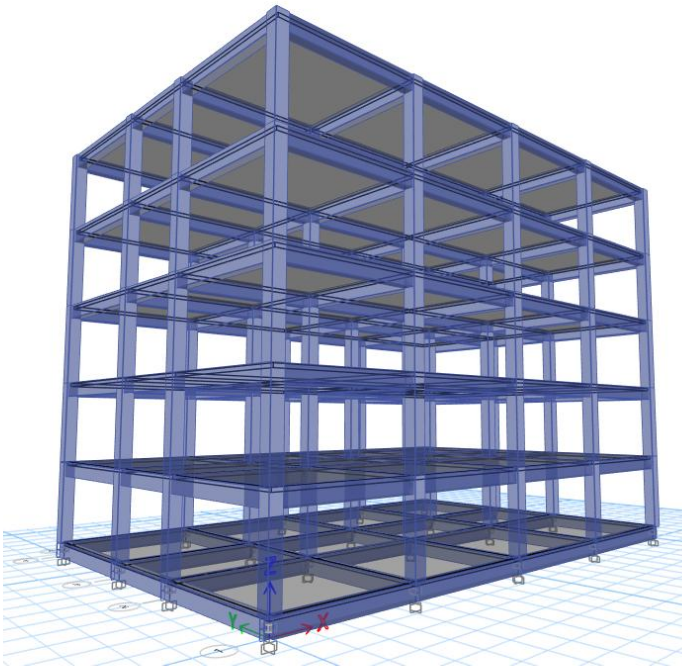


Figure III-3: Three-dimensional view of the isolated structure model.

III.5 Seismic Spectra

This section describes the response spectra used in the conventional and isolated models. Three types of spectra are considered: the Turkish elastic spectrum, the Chilean elastic spectrum, and the seismic base isolation spectrum. It is important to highlight that efforts were made to ensure that the three spectra exhibited the most similar characteristics possible, using the spectrum provided by the Turkish team as a reference. This approach was adopted to ensure a proper comparison for the design of the case study building.

III.5.1 Turkish Elastic Spectrum

The Turkish team assumed that the structure is located on moderately firm to firm sand layers, or very rigid clay, with an average shear wave velocity, V_{s30}, between 180 and 360 m/s to a depth of 30 m. Additionally, the Turkish team defined the seismic hazard as a 10% exceedance probability in 50 years, with a return period of 475 years, corresponding to a DD-2 type seismic ground motion classification. The proposed seismicity geographic coordinates (40.8507°N, 29.4073°E) correspond to a location in Turkey, specifically in the city of Izmit, an area known for its significance in seismic engineering due to its proximity to the North Anatolian Fault. The importance factor (I) assigned to the structure is 1.0.

To compare seismic designs between different countries, the Turkish team developed a pseudo-acceleration elastic spectrum based on the aforementioned characteristics, using the TBDY (2018) standard as a reference. This spectrum will serve as a model to generate the other spectra, aiming to achieve the highest possible similarity and equivalence. Figure III-4 presents the elastic spectrum provided by the Turkish team, highlighting its most representative values.

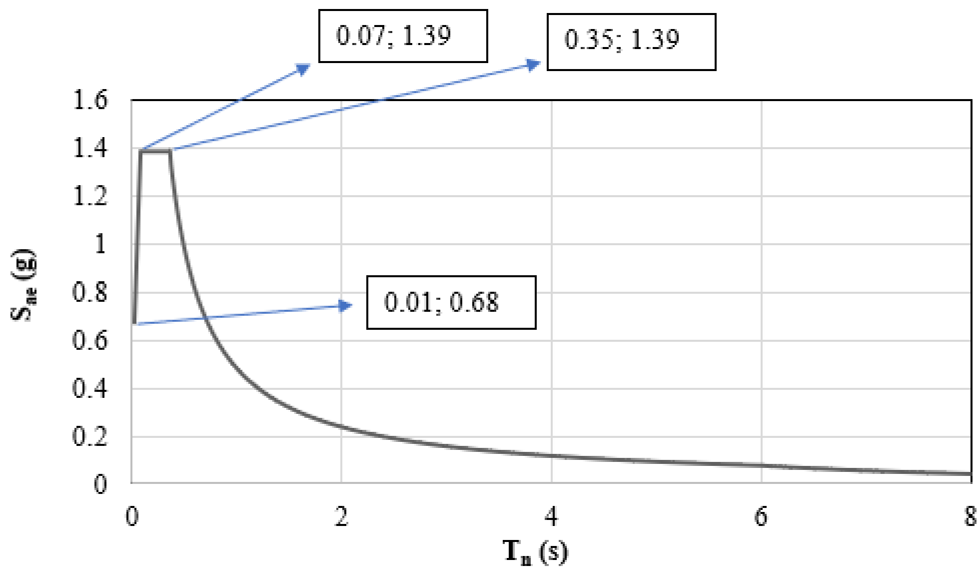


Figure III-4: Pseudo-acceleration elastic spectrum provided by the Turkish team.

III.5.2 Chilean Elastic Spectrum

The Chilean elastic spectrum was generated according to the requirements of the NCh433 (2009) standard. To achieve a shape and characteristics as similar as possible to the spectrum provided by the Turkish team, Type D soil was considered. Additionally, an occupancy category II, corresponding to residential buildings, was adopted, with an importance factor I = 1.0. The seismic zone was defined as Type 3, with an effective acceleration A0 = 0.40g. Figure III-5 compares the Chilean elastic spectrum based on NCh433 (2009) and the Turkish elastic spectrum based on TBDY (2018), highlighting the most representative values of the generated Chilean spectrum.

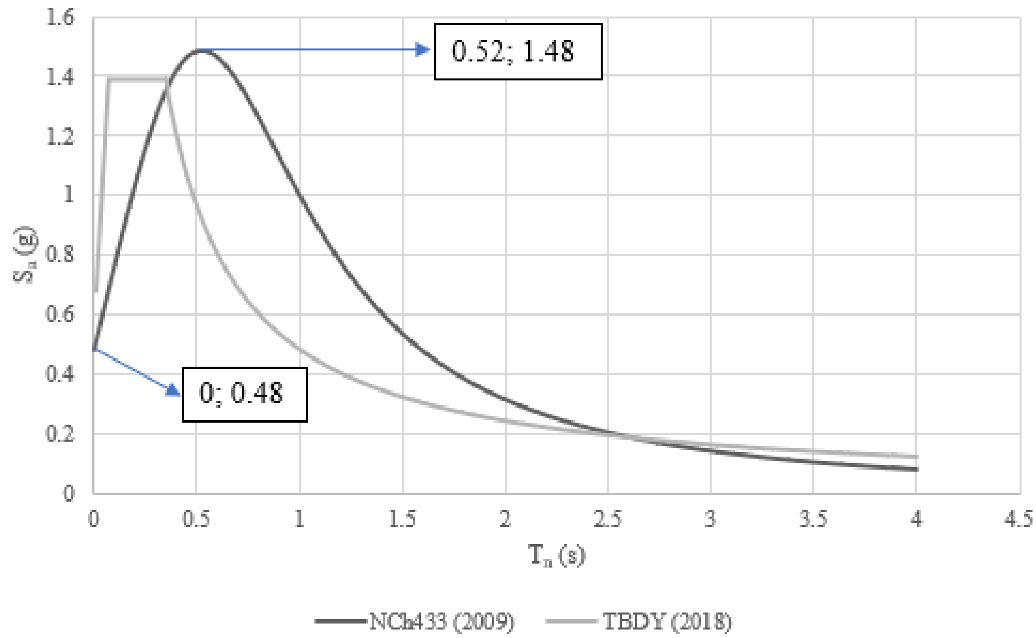


Figure III-5: Comparison between the Chilean elastic spectrum based on NCh433 (2009) and the Turkish elastic spectrum based on TBDY (2018).

III.5.3 Chilean Base Spectrum for Seismic Isolation

According to the requirements of the Chilean standard NCh2745 (2013), a base spectrum was generated for the design of the isolated structure. This spectrum was defined considering the same soil type (Type D, or III according to NCh2745) and the same seismic zoning (Type 3) used in the Chilean elastic spectrum for the conventional structure. Figure III-6 compares the Chilean and Turkish elastic spectra of the conventional structure with the generated base isolation spectrum, highlighting the most representative values of the latter.

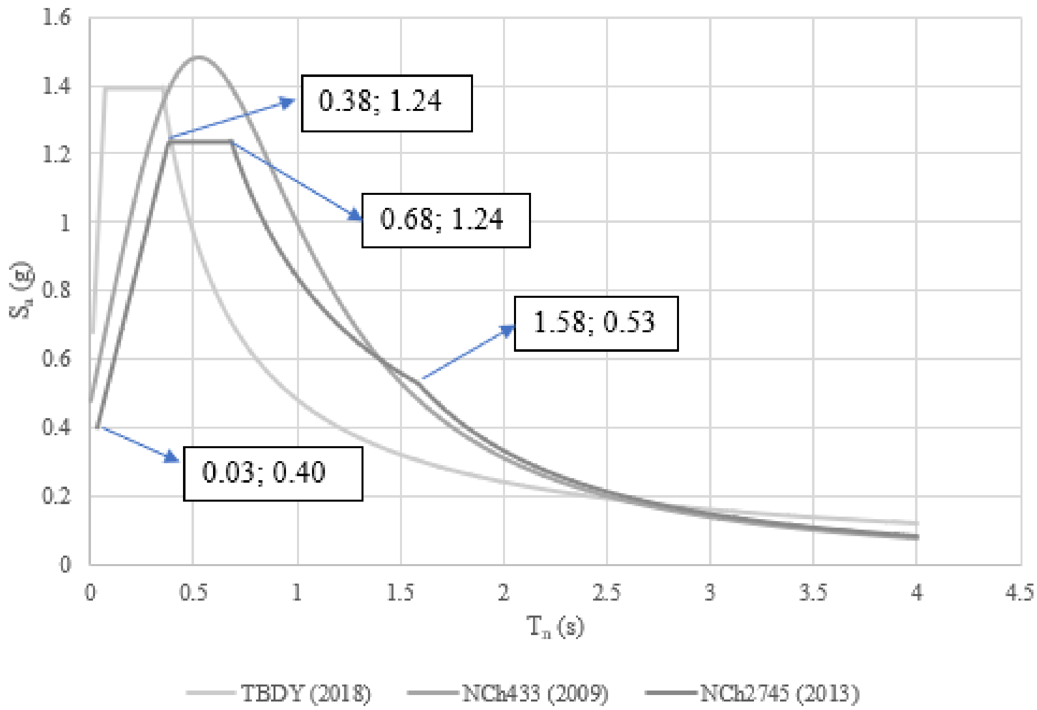


Figure III-6: Comparison between the base isolation spectrum, generated according to NCh2745 (2013), and the Chilean elastic spectrum, based on NCh433 (2009), and the Turkish elastic spectrum, based on TBDY (2018).

III.6 Reinforced Concrete Elements

The current Chilean standard regulating the design and calculation requirements for reinforced concrete structures is NCh430 (2008), which is referenced by NCh433 (2009) and NCh2745 (2013). In Clause 3, NCh430 (2008) establishes the adoption of the provisions of the ACI 318-05 Code as the basis for the design and calculation of reinforced concrete structures. In this study, it was decided to use ACI 318-19, to consider the most recent advances in structural knowledge. This code introduces substantial improvements in seismic design requirements, inelastic behavior, and structural performance criteria, ensuring a safer, more efficient solution that is adapted to current seismic demands. It is important to highlight that this choice does not compromise the fundamental principles of NCh430 (2008), as the more recent version of ACI 318 retains its essential guidelines while incorporating adjustments and improvements derived from ongoing research and experience in the field of structural engineering.

The design of the structural elements of the case study building is organized into four main groups: slabs, beams, columns, and joints; the design of these elements applies to all three analyzed models. It is important to note that, according to sections 21.2.1.2 and 21.2.1.4 of NCh430 (2008), the design of seismic-resistant reinforced concrete elements throughout Chile must consider a high seismic risk level. Furthermore, in the case of buildings structured solely with frames, these must be designed as special moment-resisting frames (SMF), following the provisions of ACI 318.

III.6.1 Design of Slabs

The design of solid reinforced concrete slabs in two directions according to ACI 318-19 begins with the verification of the minimum required thickness according to Chapter 7 (7.3.1), considering the support and load conditions. The calculation of flexural reinforcement is performed according to Chapter 8 (8.6 for ultimate moments and 8.4 for reinforcement design). Punching shear resistance is checked according to Chapter 22 (22.6 and 22.6.5 for additional reinforcement if necessary). The reinforcement detailing must comply with the requirements of Chapter 25 (25.4 for development lengths and 25.7 for slabs in two directions). Finally, deflections and cracking control are evaluated according to Chapter 24 (24.2 for displacements and 24.3 for cracking).

III.6.2 Design of Beams

For the design of SMF beams according to ACI 318-19, the moment capacity must be established according to Chapter 8 (8.4 for capacity calculation), and the flexural reinforcement is verified according to Chapter 9 (9.3 for minimum and maximum reinforcement ratios). For seismic-resistant design, Chapter 18 regulates the behavior of special frames, particularly Section 18.6, which establishes minimum dimensions, ductility zone lengths, and transverse reinforcement detailing. The shear is checked following Chapter 22 (22.5 for shear in beams), ensuring adequate capacity to resist forces induced by the plastic moment. The reinforcement detailing is governed by Chapter 25 (25.4 for anchorage and development lengths, and 25.7 for proper confinement in critical zones), ensuring ductile behavior of SMF beams.

III.6.3 Design of Columns

The design of SMF columns according to ACI 318-19 begins with the evaluation of axial and moment resistance according to Chapter 8 (8.4 for moment-axial interaction) and Chapter 9 (9.3 for minimum reinforcement ratios for longitudinal and transverse reinforcement). For seismic-resistant design, Chapter 18 is essential, especially Section 18.7, which establishes requirements for columns in special frames, such as the Strong Column – Weak Beam relation ($\sum M_{nc} \geq 1.2 \sum M_{nb}$). The shear is verified according to Chapter 22 (22.5

for shear resistance in columns). The reinforcement detailing is designed following Chapter 25 (25.4 for development and anchorage lengths, and 25.7 for confinement in critical zones), ensuring the ductility and proper behavior of SMF columns during seismic events.

III.6.4 Node Verifications

The design and verification of SMF nodes according to ACI 318-19 ensure their capacity to efficiently transmit forces between beams and columns during seismic events. The shear demands in the node are evaluated according to Chapter 22 (22.6 for calculating bidirectional shear in interior, exterior, and corner nodes). The seismic behavior of nodes is regulated by Chapter 18 (18.8), which establishes requirements for resistance, detailing, and confinement for nodes in special frames. Finally, the reinforcement detailing is designed according to Chapter 25 (25.7 for transverse confinement reinforcement and 25.4 for anchorage lengths of longitudinal reinforcement), ensuring the structural integrity and proper performance of the nodes.

IV DESIGN OF THE CONVENTIONAL STRUCTURE

In this chapter, the results of the structural design of the conventional structure are presented, considering the seismic spectra of Chile and Turkey. The key results of the response spectra analysis are presented, which will serve as the basis for the design of the reinforced concrete elements. Finally, a comparative analysis is conducted between the results obtained for both spectra.

IV.1 Structural Design with Chilean Spectrum

IV.1.1 Modal Analysis

The parameters evaluated in the modal analysis are related to the vibratory behavior of the structure under external excitation. These parameters include the fundamental periods and the modal participation percentages of the structure. Table IV-1 presents these results, highlighting that these values are the final outcome of an iterative process carried out to meet the requirements of the NCh433 standard. As a result of this process, the following final structural dimensions were defined:

- Slab thickness: 0.12 m
- Beams: 0.45 m × 0.60 m
- Columns: 0.55 m × 0.55 m

Table IV-1: Periods and modal participation ratios of the conventional structure with Chilean spectrum.

Mode	T (s)	UX	UY	RZ
1	0.40	0.84	0.00	0.00
2	0.40	0.00	0.84	0.00
3	0.36	0.00	0.00	0.84
4	0.13	0.10	0.00	0.00
5	0.13	0.00	0.11	0.00
6	0.12	0.00	0.00	0.10
7	0.07	0.04	0.00	0.00
8	0.07	0.00	0.04	0.00
9	0.07	0.00	0.00	0.04
10	0.05	0.02	0.00	0.00
11	0.05	0.00	0.02	0.00
12	0.05	0.00	0.00	0.01
	Σ	**1.00**	**1.00**	**1.00**

IV.1.2 Design Spectrum

To obtain the design spectrum, the elastic spectrum described in section III.5.2 was used, along with the fundamental periods in the X and Y directions from the analysis. For the reinforced concrete frame system, the values R=7 and R_0 =1 were considered, respectively. With these parameters, the reduction factor R* is determined using the following expression:

$$R_x^* = R_y^* = 1 + \frac{T_x^*}{0{,}10T_0 + \frac{T_x^*}{R_0}} = 1 + \frac{0{,}40\ s}{0{,}10 * 0.75\ s + \frac{0.40\ s}{11}} = 4.60 \qquad \text{(IV.1)}$$

By dividing the elastic spectrum by R*, the reduced spectrum is obtained for each direction of analysis, and the response spectrum analysis proceeds. Figure IV-1 compares the elastic and design spectra in both directions of analysis.

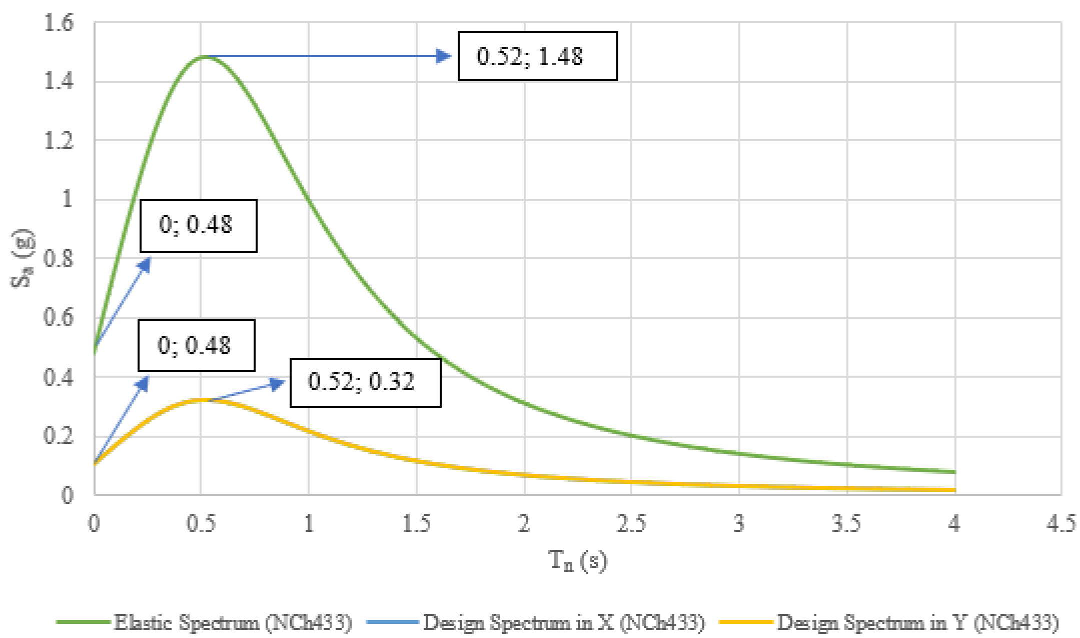


Figure IV-1: Elastic and design spectra in the X and Y directions for the conventional structure with the Chilean spectrum.

IV.1.3 Results of the Response Spectrum Analysis

a) Story Shears

It is crucial to control the base shear of the structure to ensure that the obtained values remain within the limits established by NCh433. Below are the expressions used to calculate the minimum and maximum values of the base shear, including the percentage of the total weight of the structure (W = 16213.90 KN) that they represent:

$$Q_{min} = C_{min} * I * P = 0,08 * 1 * 16213.9 = 1297\ KN\ (8\%) \quad (\text{IV.2})$$

$$Q_{max} = C_{max} * I * P = 0,168 * 1 * 16213.9 = 2723\ KN\ (17\%) \quad (\text{IV.3})$$

The base shears obtained from the analysis are:

- X Direction: Q_{bx} = 4152.20 KN (26%)
- Y Direction: Q_{by} = 4161.19 KN (26%)

Since these values exceed the maximum base shear limit, it is necessary to adjust them using the reduction factors R* to ensure compliance with regulatory requirements. Table IV-2, Figure IV-2 and Figure IV-3 present the comparison of story shear values, showing the results obtained with and without adjustments for both analysis directions.

Table IV-2: Story shears with and without adjustments for the conventional structure with the Chilean spectrum.

Story	**Qx (KN)**	**Adjusted Qx (KN)**	**Qy (KN)**	**Adjusted Qy (KN)**
5	830	545	836	547
4	2109	1384	2118	1387
3	3140	2060	3148	2061
2	3841	2519	3849	2519
1	4152	2724	4161	2724

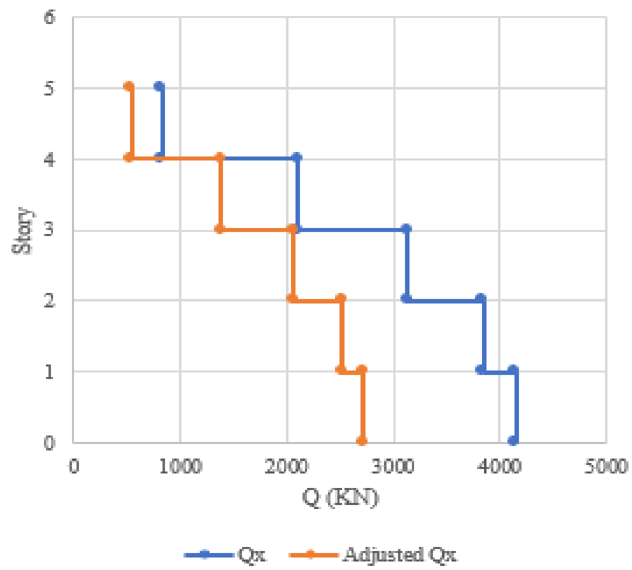


Figure IV-2: Story shear in X direction with and without adjustments for the conventional structure using the Chilean spectrum.

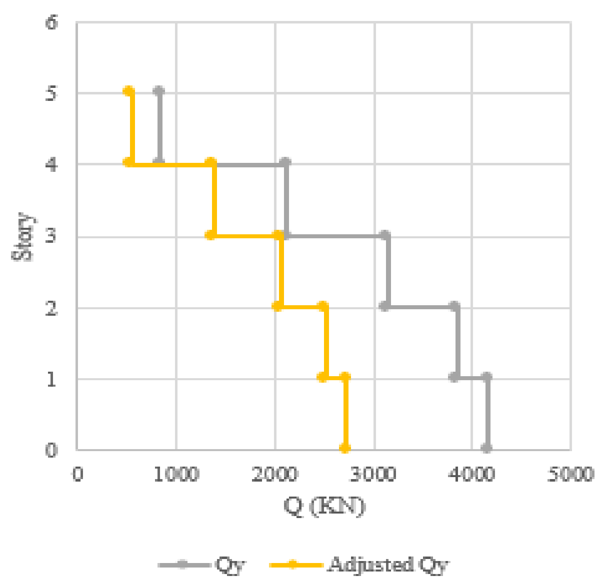


Figure IV-3: Story shear in Y direction with and without adjustments for the conventional structure using the Chilean spectrum.

b) Interstory Drifts

Regarding interstory drifts, the design was carried out in compliance with the limits set by the NCh433 standard, which states that:

- The maximum drift at the center of mass must be ≤ 0.002h.
- The difference between the drift measured at any point of the story and that at the center of mass must comply with ≤ 0.001h.

Table IV-3, Figure IV-4, and Figure IV-5 present the story drifts at the center of mass, the difference between the maximum drift and the drift at the center of mass, along with the verification of the limit established by NCh433.

Table IV-3: Verifications for interstory drifts for the conventional structure with the Chilean spectrum.

Story	Diaphragm Center Of Mass Drifts					Diaphragm Center Of Mass Drifts - Maximum Story Drifts				
	Drift Limit (NCh433)	**CM Drift (EX)**	**Verification**	**CM Drift (EY)**	**Verification**	**Drift Limit (NCh433)**	**CM Drift - Max Drift (EX)**	**Verification**	**CM Drift - Max Drift (EY)**	**Verification**
0	0.002	0	OK	0	OK	0.001	0	OK	0	OK
1	0.002	0.0012	OK	0.0011	OK	0.001	0.0001	OK	0.0002	OK
2	0.002	0.0016	OK	0.0015	OK	0.001	0.0001	OK	0.0002	OK
3	0.002	0.0013	OK	0.0013	OK	0.001	0.0001	OK	0.0002	OK
4	0.002	0.0009	OK	0.0009	OK	0.001	0.0001	OK	0.0001	OK
5	0.002	0.0005	OK	0.0005	OK	0.001	0.0000	OK	0.0001	OK

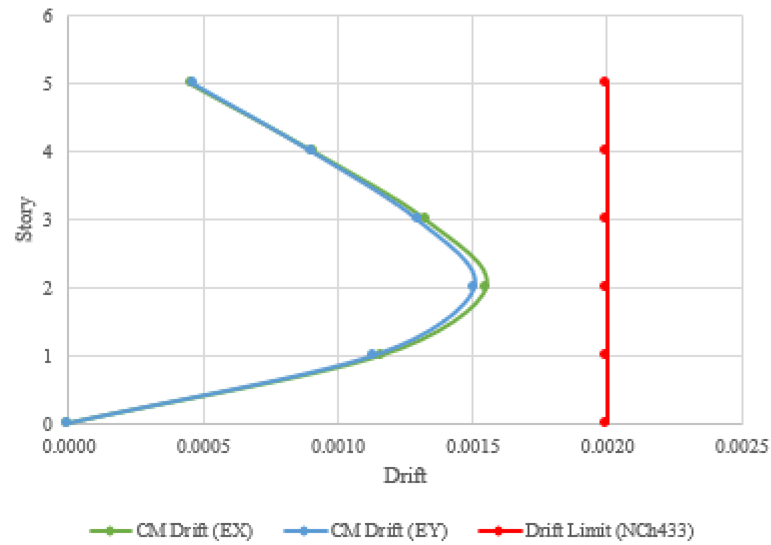


Figure IV-4: Interstory drifts at the center of mass in the two analysis directions (X and Y) for the conventional structure with the Chilean spectrum.

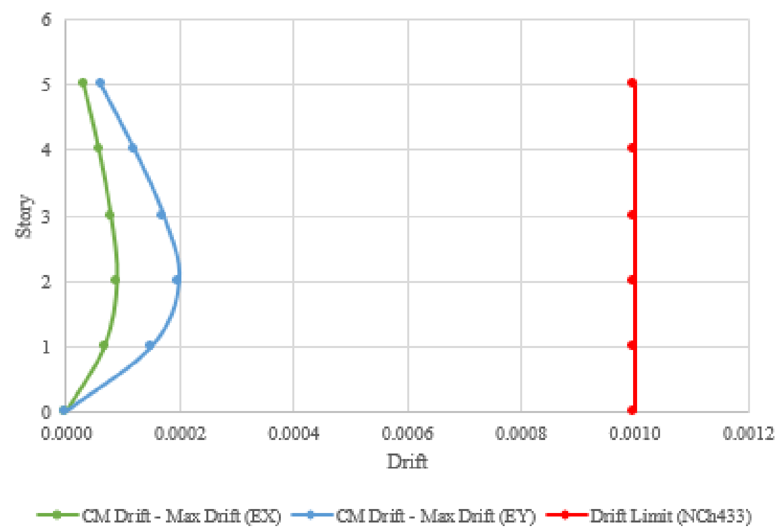


Figure IV-5: Difference between the maximum drift and the drift at the center of mass in both analysis directions (X and Y) for the conventional structure using the Chilean spectrum.

IV.1.4 Reinforced Concrete Element Design

According to the design procedure described in Section III-6 of this document, the structural elements have been designed using the final cross-sections previously mentioned. The steel reinforcement plans and construction details are presented in Annex A. Table IV-4 shows the final quantities obtained, including the concrete volume, the amount of reinforcement steel, and the volumetric ratios for each structural element.

Table IV-4: Material quantities and volumetric ratios for the conventional structure with the Chilean spectrum.

Structural Element	**Concrete (m^3)**	**Reinforcing Steel (Kg)**	**Volumetric Reinforcement Ratio, ρ_v (Kg/m^3)**
Columns	90.75	23980	264.24
Beams	176.11	25030	142.13
Slabs	132.61	12897	97.26
Σ	**399.47**	**61907**	

Table IV-5 presents a summary of the results obtained for the design of two-way slabs on all stories. Table IV-6 provides an overview of the beam design results, considering both flexural and shear design. It is important to note that four different beam types were designed for the entire structure. Finally, Table IV-7 summarizes the results for the column design, taking into account flexural-compression design, shear resistance, and verification of the strong-column weak-beam criterion. Two column types were considered for the entire structure.

Table IV-5: Slab design results for the conventional structure using the Chilean spectrum.

Story	b (cm)	h (cm)	d (cm)	ρ_{min} (%)	ρ_{max} (%)	Maximum Moments per Story, Mu (kgf-m/m)				Reinforcement Quantity, As (cm^2/m)				Flexural Reinforcement Ratio, ρ (%)				Reinforcement			
						Mu_x (+)	Mu_x (-)	Mu_y (+)	Mu_y (-)	As_x (+)	As_x (-)	As_y (+)	As_y (-)	ρ_x (+)	ρ_x (-)	ρ_y (+)	ρ_y (-)	As_x (+)	As_x (-)	As_y (+)	As_y (-)
1	100	12	9.60	0.04	2	750	1350	740	1200	2.16	3.80	2.16	3.36	0.23	0.40	0.23	0.35	Φ8c/23	Φ8c/13	Φ8c/23	Φ8c/15
2	100	12	9.60	0.04	2	748	1342	738	1192	2.16	3.77	2.16	3.34	0.23	0.39	0.23	0.35	Φ8c/23	Φ8c/13	Φ8c/23	Φ8c/15
3	100	12	9.60	0.04	2	755	1315	746	1191	2.16	3.71	2.16	3.34	0.23	0.39	0.23	0.35	Φ8c/23	Φ8c/13	Φ8c/23	Φ8c/15
4	100	12	9.60	0.04	2	765	1260	740	1060	2.16	3.53	2.16	2.95	0.23	0.37	0.23	0.31	Φ8c/23	Φ8c/14	Φ8c/23	Φ8c/17
5	100	12	9.60	0.04	2	498	805	496	730	2.16	2.23	2.16	2.16	0.23	0.23	0.23	0.23	Φ8c/23	Φ8c/22	Φ8c/23	Φ8c/23

Table IV-6: Beam design results for the conventional structure using the Chilean spectrum.

						Flexural Design										Shear Design			
Stories	Axes	Spans	b (cm)	h (cm)	d (cm)	ρ_{min} (%)	ρ_{max} (%)	Maximum Bending Moments, Mu (KN*m)		Reinforcement Quantity, As (cm^2)		Flexural Reinforcement Ratio, ρ (%)		Longitudinal Reinforcement		Maximum Shear Force (KN)	Reinforcement Quantity, Av (cm^2)	Transverse Reinforcement	
								Mu (-)	Mu (+)	As (-)	As (+)	ρ (-)	ρ (+)	As (-)	As (+)	Vu	Av	Unconfined Zone	Confined Zone
1,2,3	A, B, C, D, E	1 to 2	45	60	53.80	0.03	1.61	351.08	268.70	19.00	15.20	0.78	0.63	5φ22	4φ22	292.24	2.26	φ12@13	φ12@13
		2 to 3	45	60	53.80	0.03	1.61	353.04	313.81	19.00	19.00	0.78	0.78	5φ22	5φ22	371.67	2.26	φ12@10	φ12@10
		3 to 4	45	60	53.80	0.03	1.61	319.70	245.17	19.00	15.20	0.78	0.63	5φ22	4φ22	295.18	2.26	φ12@13	φ12@13
	1, 2, 3, 4	A to B	45	60	53.80	0.03	1.61	338.33	253.01	19.00	15.20	0.78	0.63	5φ22	4φ22	150.04	1.57	φ12@13	φ12@13
		B to C	45	60	53.80	0.03	1.61	312.83	230.46	19.00	15.20	0.78	0.63	5φ22	4φ22	150.04	1.57	φ12@13	φ12@13
		C to D	45	60	53.80	0.03	1.61	313.81	230.46	19.00	15.20	0.78	0.63	5φ22	4φ22	150.04	1.57	φ12@13	φ12@13
		D to E	45	60	53.80	0.03	1.61	311.85	237.32	19.00	15.20	0.78	0.63	5φ22	4φ22	150.04	1.57	φ12@13	φ12@13
4,5	A, B, C, D, E	1 to 2	45	60	53.80	0.03	1.61	198.09	111.80	10.16	10.16	0.42	0.42	4φ18	4φ18	196.13	1.57	φ10@14	φ10@10
		2 to 3	45	60	53.80	0.03	1.61	162.79	124.54	10.16	10.16	0.42	0.42	4φ18	4φ18	227.51	1.57	φ10@12	φ10@10
		3 to 4	45	60	53.80	0.03	1.61	176.52	107.87	10.16	10.16	0.42	0.42	4φ18	4φ18	202.02	1.57	φ10@14	φ10@10
	1, 2, 3, 4	A to B	45	60	53.80	0.03	1.61	182.40	91.20	10.16	10.16	0.42	0.42	4φ18	4φ18	194.17	1.57	φ10@13	φ10@10
		B to C	45	60	53.80	0.03	1.61	179.46	97.09	10.16	10.16	0.42	0.42	4φ18	4φ18	198.09	1.57	φ10@13	φ10@10
		C to D	45	60	53.80	0.03	1.61	179.46	97.09	10.16	10.16	0.42	0.42	4φ18	4φ18	198.09	1.57	φ10@13	φ10@10
		D to E	45	60	53.80	0.03	1.61	182.40	91.20	10.16	10.16	0.42	0.42	4φ18	4φ18	203.00	1.57	φ10@13	φ10@10

Table IV-7: Column design results for the conventional structure using the Chilean spectrum.

						Design for flexural compression					Shear Design				Strong Column – Weak Beam
Stories	Column ID	b (cm)	h (cm)	ρ_{min} (%)	ρ_{max} (%)	Maximum Axial Force (KN)	Maximum Bending Moment (KN*m)	Reinforcement Quantity (cm^2)	Flexural Reinforcement Ratio	Longitudinal Reinforcement	Maximum Shear Force (KN)	Reinforcement Quantity (cm^2)	Transverse Reinforcement		
						Pu	Mu	As	ρ (%)	As	Vu	Av	Unconfined Zone	Confined Zone	$\sum M_{nc} \geq 1.2 \sum M_{nb}$
1,2	1A, 2A, 3A, 4A, 1B, 2B, 3B, 4B, 1C, 2C, 3C, 4C, 1D, 2D, 3D, 4D, 1E, 2E, 3E, 4E	55	55	1	6	1557.29	349.11	60.82	2.01	16φ22	696.46	3.14	φ10@10	φ10@10	OK
3,4,5	1A, 2A, 3A, 4A, 1B, 2B, 3B, 4B, 1C, 2C, 3C, 4C, 1D, 2D, 3D, 4D, 1E, 2E, 3E, 4E	55	55	1	6	816.89	236.34	40.72	1.35	16φ18	488.37	3.14	φ10@10	φ10@10	OK

Figures IV-6 and IV-7 present the interaction diagrams for the flexural-compression design of the two types of columns. These diagrams show that the column strength meets the requirements of the load demands for the assigned column group of each type.

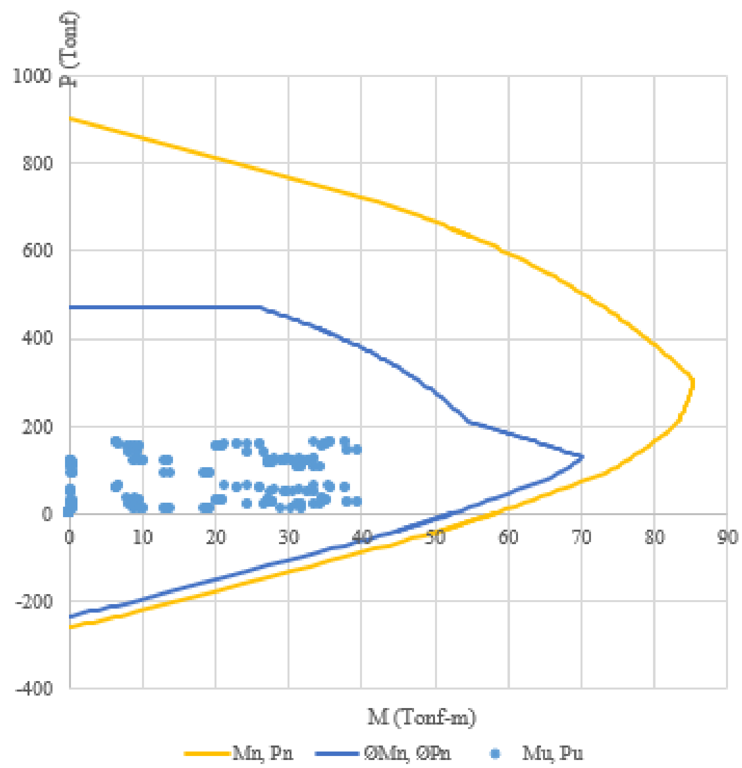


Figure IV-6: Interaction diagram and load demands for the flexural-compression design of the 55 cm x 55 cm column section with 16φ22 longitudinal reinforcement for the conventional structure using the Chilean spectrum.

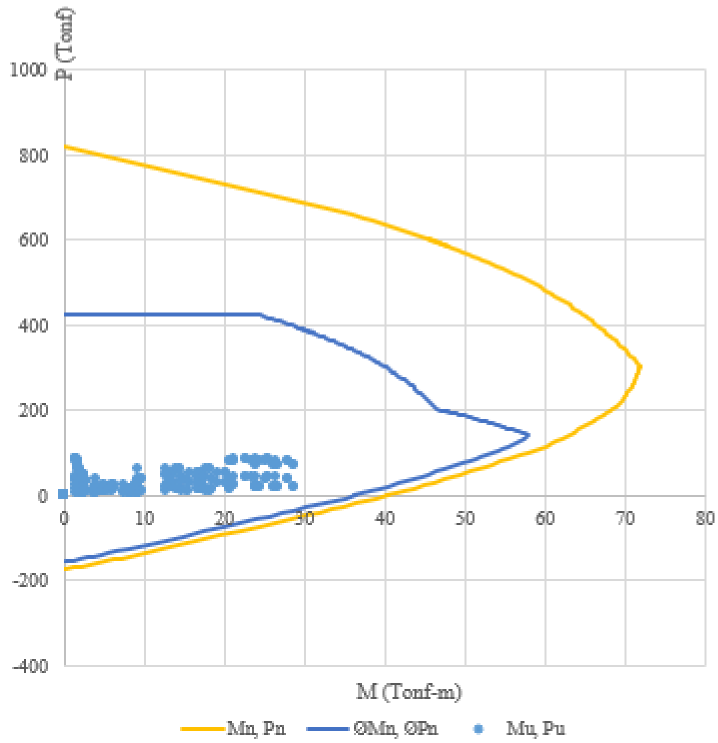


Figure IV-7: Interaction diagram and load demands for the flexural-compression design of the 55 cm x 55 cm column section with 16φ18 longitudinal reinforcement for the conventional structure using the Chilean spectrum.

IV.2 Structural Design with Turkish Spectrum

IV.2.1 Modal Analysis

In the same manner as for the model with the Chilean spectrum, the parameters of the modal analysis are obtained. Table IV-8 shows these results. After the iterative process to meet the requirements of NCh433, the following final dimensions of the structural elements were defined:

- Slab thickness: 0.12 m
- Beams: 0.30 m × 0.50 m
- Columns: 0.40 m × 0.40 m

Table IV-8: Periods and modal participation ratios of the conventional structure with the Turkish spectrum.

Mode	T (s)	UX	UY	RZ
1	0.64	0.85	0.00	0.00
2	0.63	0.00	0.85	0.00
3	0.57	0.00	0.00	0.85
4	0.21	0.10	0.00	0.00
5	0.21	0.00	0.10	0.00
6	0.18	0.00	0.00	0.10
7	0.12	0.03	0.00	0.00
8	0.12	0.00	0.03	0.00
9	0.11	0.00	0.00	0.03
10	0.09	0.01	0.00	0.00
11	0.09	0.00	0.01	0.00
12	0.08	0.00	0.00	0.01
	Σ	**1.00**	**1.00**	**1.00**

IV.2.2 Design Spectrum

To obtain the design spectrum, the elastic spectrum described in Section III.5.1 was used, along with the fundamental periods from the analysis in the X and Y directions. Since the goal is to apply Chilean practices to the Turkish spectrum, it is assumed that this spectrum adopts the same parameters as the Chilean spectrum, and the reduction factors established by NCh433 are applied. With these parameters, the reduction factor R* is determined using the following expressions:

$$R_x^* = 1 + \frac{T_x^*}{0{,}10T_0 + \frac{T_x^*}{R_0}} = 1 + \frac{0{,}64\ s}{0{,}10 * 0.75\ s + \frac{0.64\ s}{11}} = 5.80 \qquad (\text{IV.4})$$

$$R_y^* = 1 + \frac{T_y^*}{0{,}10T_0 + \frac{T_y^*}{R_0}} = 1 + \frac{0{,}63\ s}{0{,}10 * 0.75\ s + \frac{0.63\ s}{11}} = 5.77 \qquad (\text{IV.5})$$

Dividing the elastic spectrum by R*, the reduced spectrum for each analysis direction is obtained, and the response spectrum analysis can proceed. Figure IV-8 compares the elastic and design spectra in both analysis directions.

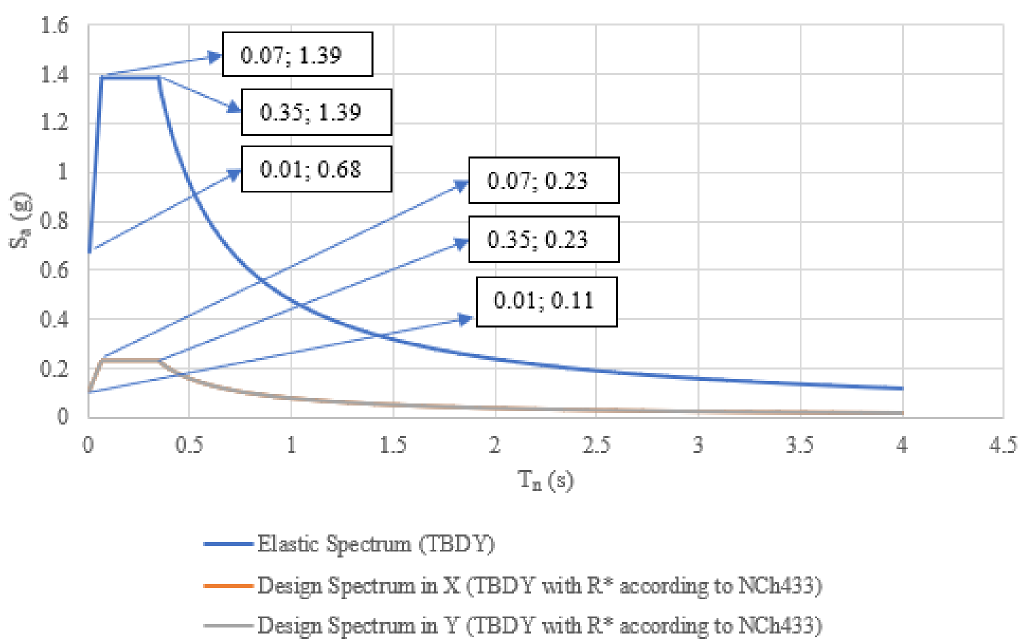


Figure IV-8: Elastic and design spectra in the X and Y directions for the conventional structure with the Turkish spectrum.

IV.2.3 Results of the Response Spectrum Analysis

a) Story Shears

The following expressions calculate the minimum and maximum base shear values for the structure according to NCh433, considering the percentage of these values relative to the weight of the structure (W = 13331.40 KN):

$$Q_{min} = C_{min} * I * P = 0{,}08 * 1 * 13331 = 1067\ KN\ (8\%) \qquad (\text{ IV.6 })$$

$$Q_{max} = C_{max} * I * P = 0{,}168 * 1 * 13331 = 2240\ KN\ (17\%) \qquad (\text{ IV.7 })$$

The base shear values obtained from the analysis are:

- X Direction: Q_{bx} = 1496 KN (11%)
- Y Direction: Q_{by} = 1517 KN (11%)

The base shears do not exceed the allowed limits; therefore, no adjustments are necessary. Table IV-9, Figure IV-9, and Figure IV-10 present the story shears for both analysis directions.

Table IV-9: Story shear forces for the conventional structure with Turkish spectrum.

Story	Qx (KN)	Qy (KN)
5	330	334
4	780	791
3	1106	1122
2	1350	1370
1	1496	1518

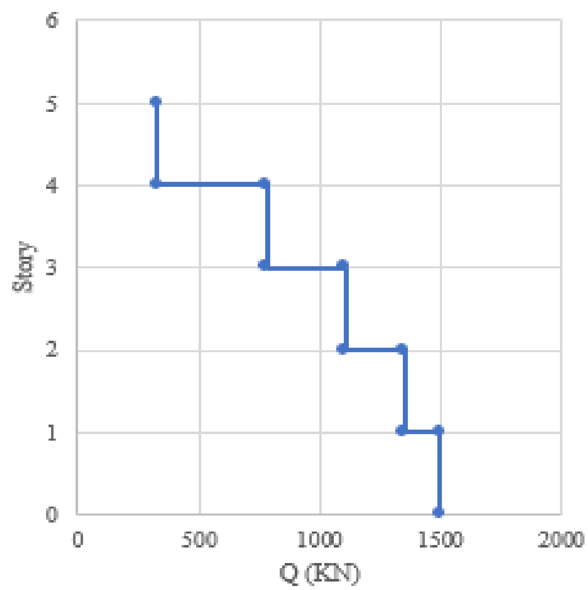


Figure IV-9: Story shear forces in X direction for the conventional structure with Turkish spectrum.

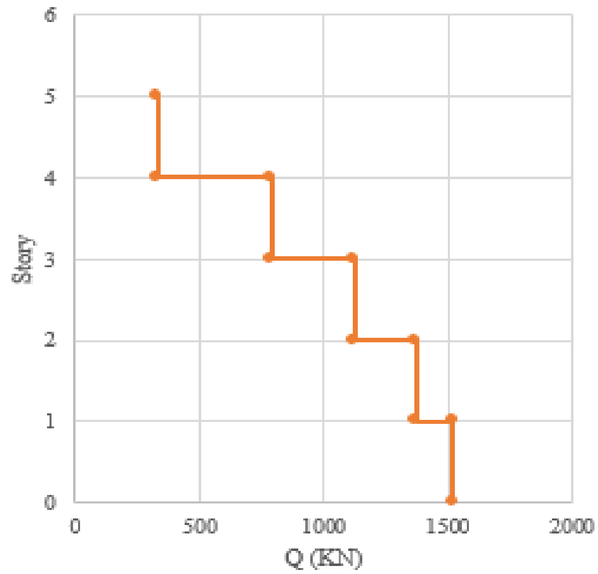


Figure IV-10: Story shear forces in Y direction for the conventional structure with Turkish spectrum.

b) Interstory Drifts

Regarding the interstory drifts, the design was carried out in compliance with the limits established in the NCh433 standard. Table IV-10, Figure IV-11, and Figure IV-12 show the interstory drifts at the center of mass, the difference between the maximum drift and the drift at the center of mass, along with the verification of the established limits.

Table IV-10: Verification of interstory drifts for the conventional structure using the Turkish spectrum.

Story	Diaphragm Center of Mass Drifts					Diaphragm Center of Mass Drifts - Maximum Story Drifts				
	Drift Limit (NCh433)	CM Drift (EX)	Verification	CM Drift (EY)	Verification	Drift Limit (NCh433)	CM Drift - Max Drift (EX)	Verification	CM Drift - Max Drift (EY)	Verification
0	0.002	0	OK	0.0000	OK	0.001	0	OK	0	OK
1	0.002	0.0013	OK	0.0013	OK	0.001	0.0001	OK	0.0002	OK
2	0.002	0.0016	OK	0.0016	OK	0.001	0.0001	OK	0.0002	OK
3	0.002	0.0014	OK	0.0014	OK	0.001	0.0001	OK	0.0002	OK
4	0.002	0.0009	OK	0.0009	OK	0.001	0.0001	OK	0.0002	OK
5	0.002	0.0004	OK	0.0004	OK	0.001	0.0001	OK	0.0001	OK

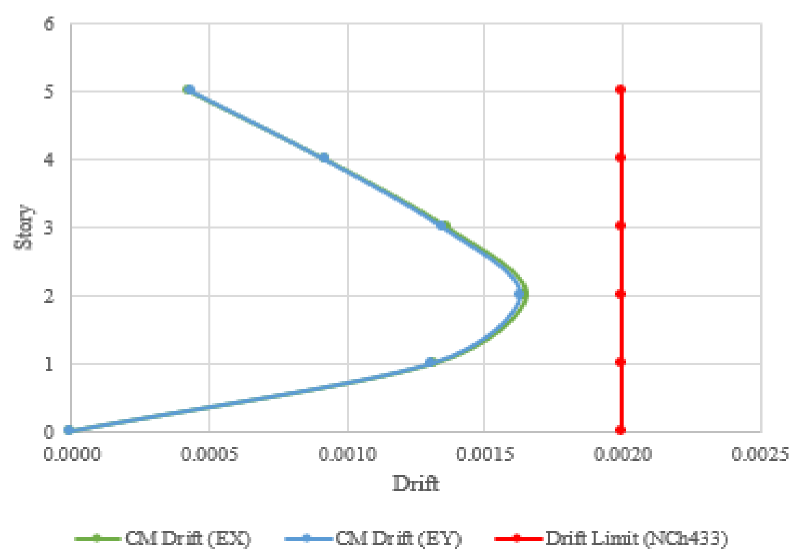


Figure IV-11: Interstory drifts at the center of mass in the two analysis directions (X and Y) for the conventional structure using the Turkish spectrum.

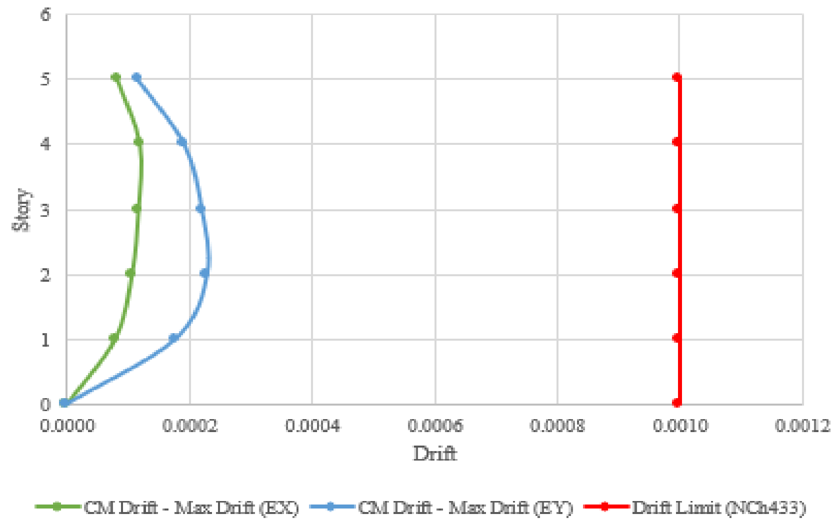


Figure IV-12: Difference between the maximum drift and the drift at the center of mass in the two analysis directions (X and Y) for the conventional structure using the Turkish spectrum.

IV.2.4 Design of Reinforced Concrete Elements

Following the procedure described in Section III-6, the structural elements were designed using the previously established final sections. The steel reinforcement plans and corresponding construction details are presented in Annex B. Table IV-11 summarizes the final quantities obtained, detailing the volume of concrete, the amount of reinforcing steel, and the volumetric reinforcement ratios for each structural element.

Table IV-11: Material quantities and volumetric reinforcement ratios for the conventional structure with Turkish spectrum.

Structural Element	**Concrete (m^3)**	**Reinforcing Steel (Kg)**	**Volumetric Reinforcement Ratio, ρ_v (Kg/m^3)**
Columns	48.00	16040	334.17
Beams	101.33	15966	157.56
Slabs	142.06	12410	87.36
Σ	**291.39**	**44416**	

Table IV-12 summarizes the design results of the two-way slabs for all stories. Table IV-13 details the beam design results, considering both bending and shear, with four types of beams defined for the structure. Finally, Table IV-14 presents the column design results, including flexural compression, shear resistance, and verification of the strong-column weak-beam criterion, with two types of columns distributed according to the required loads and stiffness.

Table IV-12: Slab design results for the conventional structure using the Turkish spectrum.

Story	b (cm)	h (cm)	d (cm)	ρ_{min} (%)	ρ_{max} (%)	Maximum Moments per Story, Mu (kgf-m/m)				Reinforcement Quantity, As (cm^2/m)				Flexural Reinforcement Ratio, ρ (%)				Reinforcement			
						Mu_x (+)	Mu_x (-)	Mu_y (+)	Mu_y (-)	As_x (+)	As_x (-)	As_y (+)	As_y (-)	ρ_x (+)	ρ_x (-)	ρ_y (+)	ρ_y (-)	As_x (+)	As_x (-)	As_y (+)	As_y (-)
1	100	12	9.60	0.04	2	750	1350	740	1200	2.16	3.80	2.16	3.36	0.23	0.40	0.23	0.35	Φ8c/23	Φ8c/13	Φ8c/23	Φ8c/15
2	100	12	9.60	0.04	2	748	1342	738	1192	2.16	3.77	2.16	3.34	0.23	0.39	0.23	0.35	Φ8c/23	Φ8c/13	Φ8c/23	Φ8c/15
3	100	12	9.60	0.04	2	755	1315	746	1191	2.16	3.71	2.16	3.34	0.23	0.39	0.23	0.35	Φ8c/23	Φ8c/13	Φ8c/23	Φ8c/15
4	100	12	9.60	0.04	2	765	1260	740	1060	2.16	3.53	2.16	2.95	0.23	0.37	0.23	0.31	Φ8c/23	Φ8c/14	Φ8c/23	Φ8c/17
5	100	12	9.60	0.04	2	498	805	496	730	2.16	2.23	2.16	2.16	0.23	0.23	0.23	0.23	Φ8c/23	Φ8c/22	Φ8c/23	Φ8c/23

Table IV-13: Beam design results for the conventional structure using the Turkish spectrum.

Stories	Axes	Spans	b (cm)	h (cm)	d (cm)	ρ_{min} (%)	ρ_{max} (%)	Flexural Design: Maximum Bending Moments, Mu (KN*m)		Flexural Design: Reinforcement Quantity, As (cm^2)		Flexural Design: Flexural Reinforcement Ratio, ρ (%)		Flexural Design: Longitudinal Reinforcement		Shear Design: Maximum Shear Force (KN)	Shear Design: Reinforcement Quantity, Av (cm^2)	Shear Design: Transverse Reinforcement	
								Mu (-)	Mu (+)	As (-)	As (+)	ρ (-)	ρ (+)	As (-)	As (+)	Vu	Av	Unconfined Zone	Confined Zone
1,2,3	A, B, C, D, E	1 to 2	30	50	45.10	0.03	1.61	214.76	152.98	15.20	10.16	1.12	0.75	6φ18	4φ18	187.31	1.57	φ10@10	φ10@10
		2 to 3	30	50	45.10	0.03	1.61	203.00	164.75	15.20	12.70	1.12	0.94	6φ18	5φ18	216.73	1.57	φ10@10	φ10@10
		3 to 4	30	50	45.10	0.03	1.61	197.11	125.52	15.20	10.16	1.12	0.75	6φ18	4φ18	186.33	1.57	φ10@10	φ10@10
	1, 2, 3, 4	A to B	30	50	45.10	0.03	1.61	203.98	144.16	15.24	10.16	1.13	0.75	6φ18	4φ18	179.46	1.57	φ10@10	φ10@10
		B to C	30	50	45.10	0.03	1.61	183.38	107.87	15.24	7.62	1.13	0.56	6φ18	3φ18	163.77	1.57	φ10@13	φ10@10
		C to D	30	50	45.10	0.03	1.61	183.38	107.87	15.24	7.62	1.13	0.56	6φ18	3φ18	164.75	1.57	φ10@13	φ10@10
		D to E	30	50	45.10	0.03	1.61	198.09	119.64	15.24	10.16	1.13	0.75	6φ18	4φ18	174.56	1.57	φ10@10	φ10@10
4,5	A, B, C, D, E	1 to 2	30	50	45.10	0.03	1.61	136.31	62.76	10.16	5.08	0.75	0.38	4φ18	2φ18	134.35	1.57	φ10@16	φ10@10
		2 to 3	30	50	45.10	0.03	1.61	111.80	76.49	7.62	5.08	0.56	0.38	3φ18	2φ18	125.52	1.57	φ10@18	φ10@10
		3 to 4	30	50	45.10	0.03	1.61	121.60	60.80	10.16	5.08	0.75	0.38	4φ18	2φ18	138.27	1.57	φ10@16	φ10@10
	1, 2, 3, 4	A to B	30	50	45.10	0.03	1.61	128.47	64.23	10.16	5.08	0.75	0.38	4φ18	2φ18	132.39	1.57	φ10@16	φ10@10
		B to C	30	50	45.10	0.03	1.61	120.62	60.31	7.62	5.08	0.56	0.38	3φ18	2φ18	123.56	1.57	φ10@18	φ10@10
		C to D	30	50	45.10	0.03	1.61	120.62	60.31	7.62	5.08	0.56	0.38	3φ18	2φ18	122.58	1.57	φ10@18	φ10@10
		D to E	30	50	45.10	0.03	1.61	128.47	64.23	10.16	5.08	0.75	0.38	4φ18	2φ18	138.27	1.57	φ10@16	φ10@10

Table IV-14: Column design results for the conventional structure using the Turkish spectrum.

Stories	Column ID	b (cm)	h (cm)	ρ_{min} (%)	ρ_{max} (%)	Design for flexural compression					Shear Design				Strong Column – Weak Beam
						Maximum Axial Force (KN)	Maximum Bending Moment (KN*m)	Reinforcement Quantity (cm^2)	Flexural Reinforcement Ratio	Longitudinal Reinforcement	Maximum Shear Force (KN)	Reinforcement Quantity (cm^2)	Transverse Reinforcement		
						Pu	Mu	As	ρ (%)	As	Vu	Av	Unconfined Zone	Confined Zone	$\sum M_{nc} \geq 1.2 \sum M_{nb}$
1,2	1A, 2A, 3A, 4A, 1B, 2B, 3B, 4B, 1C, 2C, 3C, 4C, 1D, 2D, 3D, 4D, 1E, 2E, 3E, 4E	40	40	1	6	1315.07	187.31	45.61	2.85	12φ22	301.75	3.14	φ10@13	φ10@10	OK
3,4,5	1A, 2A, 3A, 4A, 1B, 2B, 3B, 4B, 1C, 2C, 3C, 4C, 1D, 2D, 3D, 4D, 1E, 2E, 3E, 4E	40	40	1	6	766.88	150.04	30.54	1.91	12φ18	226.34	3.14	φ10@10	φ10@10	OK

Figures IV-13 and IV-14 show the interaction diagrams for the flexural compression design of the two column types. These graphs demonstrate that the load-bearing capacity of the columns meets the required demands for each assigned group, thus validating their compliance with the established structural requirements.

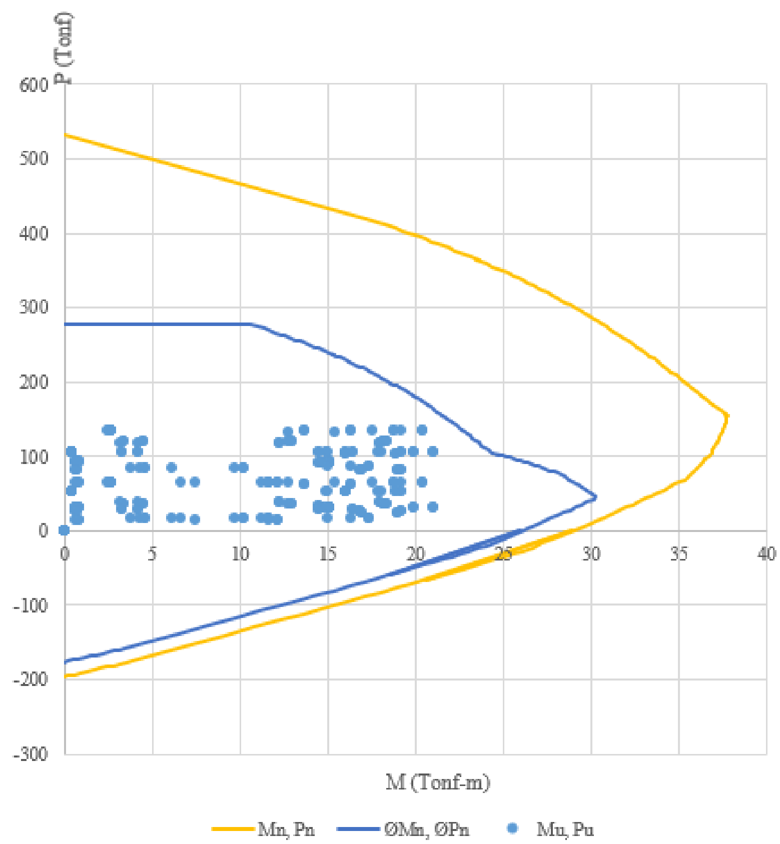


Figure IV-13: Interaction diagram and design loads for the flexural compression design of the 40 cm x 40 cm column section with 12φ22 longitudinal reinforcement for the conventional structure with the Turkish spectrum.

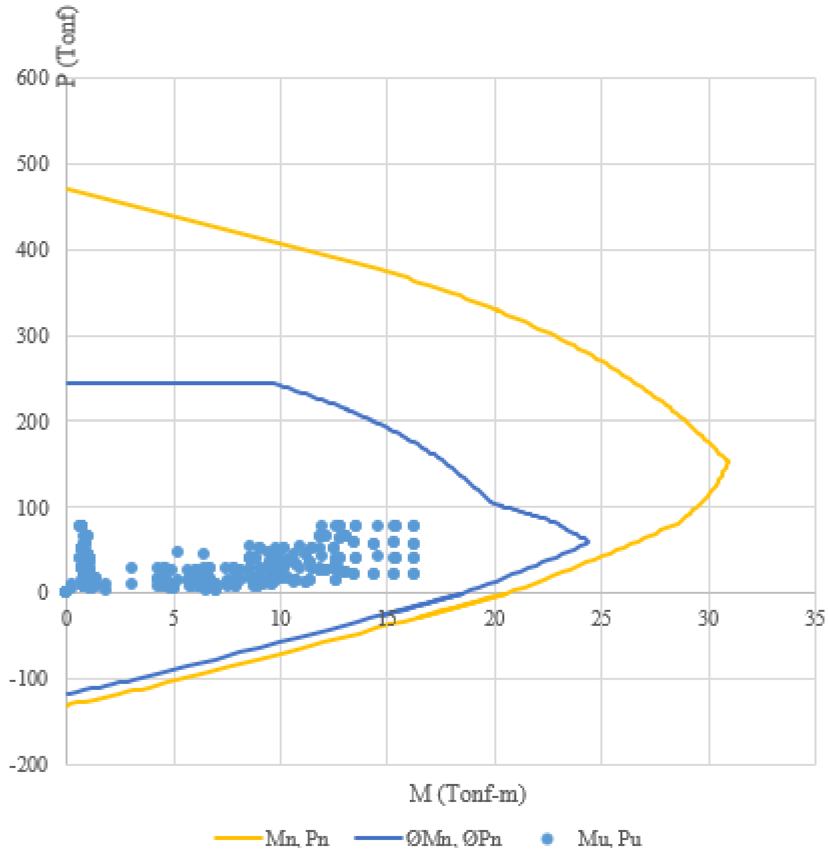


Figure IV-14: Interaction diagram and design loads for the flexural compression design of the 40 cm x 40 cm column section with 12φ18 longitudinal reinforcement for the conventional structure with the Turkish spectrum.

IV.3 Comparative Analysis

This section presents a comparative analysis of the results obtained for the conventional structure. The objective is to evaluate the differences in structural behavior and the design of the reinforced concrete elements. Table IV-15 summarizes the main results and the percentage variation of the conventional structure with the Turkish spectrum compared to the Chilean spectrum.

Table IV-15: Comparative results for the conventional structure with Chilean and Turkish spectra.

Parameters		**Conventional Structure**		**% Variation of Conventional Structure with Turkish Spectrum Compared to Chilean Spectrum**
		Chilean Spectrum	**Turkish Spectrum**	
Slabs	h(m)	0.12	0.12	0%
Beams	b(m)	0.45	0.30	-33%
	h (m)	0.60	0.50	-17%
Columns	b(m)	0.55	0.40	-27%
	h (m)	0.55	0.40	-27%
Structure Weight	W (KN)	16214	13331	-18%
Fundamental Periods	T_x (s)	0.40	0.64	58%
	T_y (s)	0.40	0.63	58%
Design pseudo-acceleration	Sa_x (g)	0.31	0.13	-58%
Reduction Factors	R^*_x	4.62	5.80	26%
	R^*_y	4.59	5.77	26%
Base Shear	Q_x (KN)	2724	1496	-45%
	Q_y (KN)	2724	1518	-44%
Flexural Reinforcement Ratio (Max)	ρ_{slab} (%)	0.40	0.40	0%
	ρ_{beam} (%)	0.78	1.13	45%
	ρ_{col} (%)	2.01	2.85	42%
Concrete Volume	V_c (m^3)	399	291	-27%
Reinforcing Steel Quantity	W_s (Kg)	61907	44416	-28%
Volumetric Reinforcement Ratio	$\rho_{v,\,slab}$ (Kg/m^3)	97	87	-10%
	$\rho_{v,\,beam}$ (Kg/m^3)	142	158	11%
	$\rho_{v,\,col}$ (Kg/m^3)	264	334	26%

Figure IV-15 presents a comparison between the Chilean and Turkish design spectra. Additionally, each spectrum indicates the period corresponding to the first vibration mode of each structure, along with the pseudo-acceleration associated with that period.

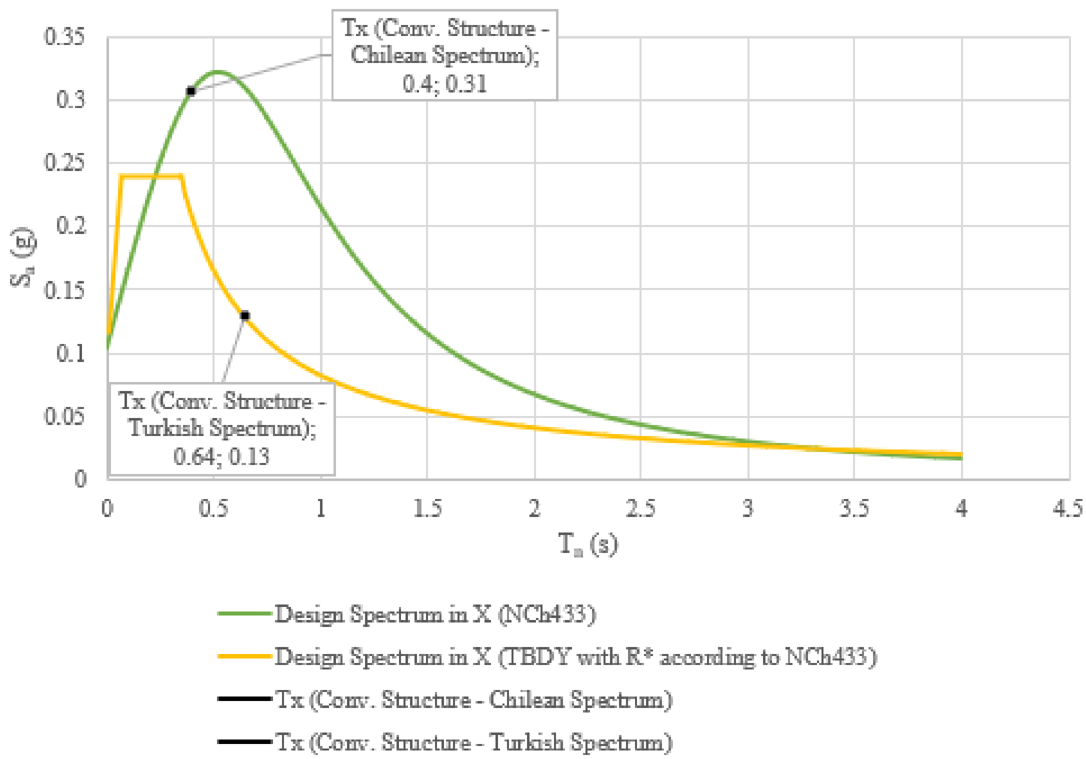


Figure IV-15: Comparison of the Chilean and Turkish design spectra for the conventional structure, including the period and pseudo-acceleration corresponding to the first vibration mode of each structure.

The obtained results indicate that the structure designed using the Chilean spectrum experiences higher demands compared to the one designed with the Turkish spectrum. This is mainly due to the fact that the pseudo-acceleration corresponding to the first mode in the

Chilean spectrum (0.31g) is significantly higher than in the Turkish spectrum (0.13g), representing a 58% reduction. In particular, the dimensions of the structural elements, especially beams and columns, are 17% to 33% smaller in the case of the Turkish spectrum. Additionally, the total weight of the structure is 13% lower compared to the one designed with the Chilean spectrum.

It is important to highlight that verifications were carried out to ensure compliance with the story drift limits established by the NCh433 standard. In the case of the Chilean spectrum, the initially assigned dimensions based on the Turkish spectrum design did not meet these requirements, making it necessary to increase the sections of the structural elements to comply with the regulations. From a seismic parameter’s perspective, the structure subjected to the Chilean spectrum exhibits a stiffer behavior, with a fundamental period of 0.40 s, compared to 0.64 s recorded under the Turkish spectrum. Additionally, the base shear forces are approximately 45% lower in the case of the Turkish spectrum compared to those obtained with the Chilean spectrum.

Regarding structural reinforcement, the conventional structure designed with the Turkish spectrum requires 28% less reinforcement (in kilograms) compared to the one designed with the Chilean spectrum. However, when analyzing volumetric reinforcement ratios, the structure based on the Turkish spectrum shows higher values in beams and columns, with increases of 11% and 26%, respectively. Similarly, increases in longitudinal reinforcement ratios were observed in beams and columns, with increments of 45% and 42%, respectively

V DESIGN OF THE ISOLATED STRUCTURE

This chapter presents the results of the structural design of the base-isolated structure, detailing key aspects of the process. It includes the results of the response spectrum analysis, which allows for an evaluation of the dynamic behavior of the structure, as well as the design and optimization of reinforced concrete elements. It is important to note that the design of the isolators is not part of this study; only their properties are considered to meet the defined objectives. Finally, a comparative analysis is conducted between the conventional and base-isolated structures to highlight differences in terms of performance and structural efficiency.

V.1 Modal Analysis

As in the case of the conventional structure, the modal analysis parameters were obtained for the isolated structure. However, in this case, it was necessary to incorporate the isolation level at the base of the conventional model. Table V-1 presents the periods and modal participation ratios of the isolated model. Following an iterative process to meet the requirements and limits established by NCh2745, the following final dimensions for the structural elements were determined:

- Slab thickness: 0.12 m
- Beams: 0.30 m × 0.50 m
- Columns: 0.40 m × 0.40 m

Table V-1: Periods and modal participation ratios of the isolated structure.

Mode	T (s)	UX	UY	RZ
1	2.57	1.00	0.00	0.00
2	2.57	0.00	1.00	0.00
3	2.32	0.00	0.00	1.00
4	0.36	0.00	0.00	0.00
5	0.36	0.00	0.00	0.00
6	0.32	0.00	0.00	0.00
7	0.18	0.00	0.00	0.00
8	0.18	0.00	0.00	0.00
9	0.16	0.00	0.00	0.00
10	0.12	0.00	0.00	0.00
11	0.11	0.00	0.00	0.00
12	0.10	0.00	0.00	0.00
13	0.09	0.00	0.00	0.00
14	0.08	0.00	0.00	0.00
15	0.08	0.00	0.00	0.00
16	0.07	0.00	0.00	0.00
17	0.07	0.00	0.00	0.00
18	0.06	0.00	0.00	0.00
	Σ	**1.00**	**1.00**	**1.00**

V.2 Isolation System

V.2.1 Selection Criteria for the Isolators

The selection of the seismic isolators was based on three main criteria: ease of installation, market availability and extensive research supporting their effectiveness. Among the most widely used seismic isolation devices, high-damping rubber bearings (HDRs) stand out due to their ability to combine flexibility and energy dissipation in a single element, while also being relatively easy to manufacture.

These isolators provide effective damping between 10% and 20%, significantly reducing the seismic response of the structure. Additionally, they exhibit higher stiffness during the initial loading cycles, which stabilizes from the third cycle onward, ensuring a predictable and efficient behavior under cyclic loading. For these reasons, HDR isolators were selected as the most suitable option for this design. Figure V-1 illustrates the geometry and hysteretic behavior of the force-displacement relationship characteristic of an elastomeric isolator, providing insight into its performance within the studied building.

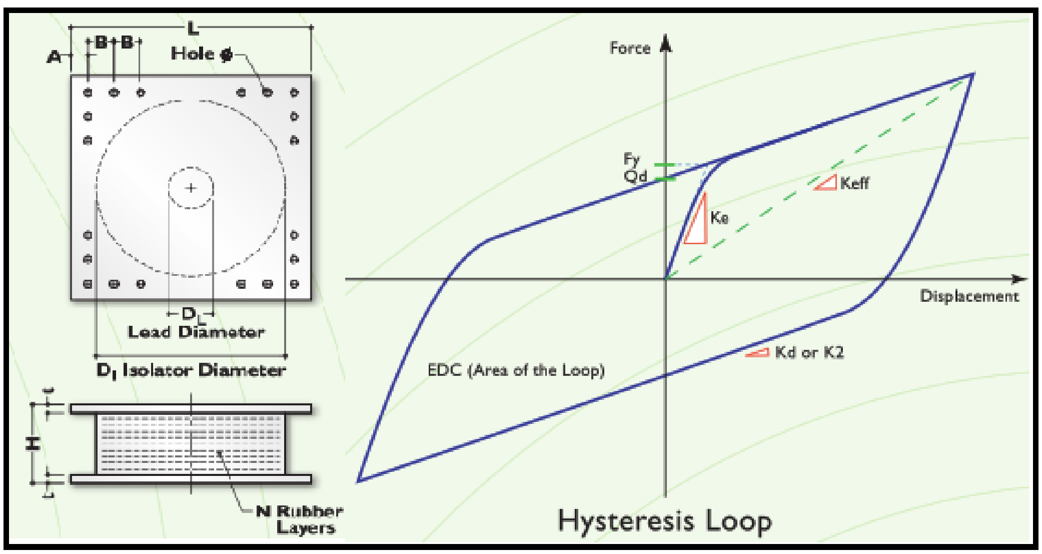


Figure V-1: Typical geometry and hysteretic behavior of an elastomeric isolator (DIS, 2007).

For this study, an effective damping of β_D = 15% and an isolator height of 0.50 m were selected. These values represent the typical average for HDR isolators, according to available technical catalogs.

V.2.2 Static Analysis

According to the recommendations of NCh2745, all seismically isolated structures, or any part thereof, must be designed and constructed to withstand at least the forces and displacements established through static analysis. The following section details the design parameter calculations necessary to meet these requirements. Table V-2 summarizes the required parameters for the subsequent calculations, considering the soil type and seismic zoning previously described.

Table V-2: Required parameters for the static analysis of the isolated structure.

Parameter	**Value**
Soil Type	III
Seismic Zone	3
Effective Damping (β_D)	15%
B_D or B_M Factor	1.67
Factor Dependent on Seismic Zone (Z)	1.25
Amplification Factor for Maximum Earthquake (M_M)	1.2
Response Reduction Factor for Superstructure Design (R_s)	2

a) Maximum and Design Displacements

The following equations calculate the design displacement (D_D) and maximum displacement (D_M), assumed to occur at the center of mass of the structural system:

$$D_D = \frac{C_D}{B_D} = \frac{330 * 1.25}{1.67} = 24.70\ cm \tag{V.1}$$

$$D_M = \frac{C_M}{B_M} = \frac{330 * 1.2 * 1.25}{1.67} = 29.64\ cm \qquad (V.2)$$

b) Effective Stiffness

To determine the effective stiffness of each isolator (K_D), it is necessary to obtain the weight of the isolated structure (including the new isolation level) and the target period to which the structure is adjusted. These values are:

- Weight of the isolated structure: W=16058 KN
- Target period: T_D = 2.50 s

The total stiffness of the isolation system is then calculated as follows:

$$K_{D,total} = \left(\frac{2\pi}{T_D}\right)^2 * \frac{W}{g} = 10340\ KN/m \qquad (V.3)$$

Considering that each column requires 20 isolators, the stiffness per isolator is:

$$K_D = \frac{K_{D,total}}{N} = \frac{10340.13}{20} = 517\ KN/m \qquad (V.4)$$

This effective stiffness value is then input into ETABS under the directional properties (X and Y) of the link element representing the isolator.

c) Minimum Lateral Forces

The minimum lateral forces for the isolation system (V_b), superstructure (V_s), and the minimum required by NCh433 ($Q_{min,\ NCh433}$) are calculated below. Additionally, the percentage of the total structural weight represented by each of these forces is determined.

$$V_b = \frac{K_{D,max} * D_D}{R_b} = \frac{10340.13 * 24.70}{1} = 2554\ KN\ (16\%) \qquad (\text{V.5})$$

$$V_S = \frac{K_{D,max} * D_D}{R_s} = \frac{10340.13 * 24.70}{2} = 1277\ KN\ (8\%) \qquad (\text{V.6})$$

$$Q_{min,NCh433} = C_{min} * I * P = 0.0667 * 1 * 16058.89 = 1071\ KN\ (7\%) \qquad (\text{V.7})$$

V.2.3 Dynamic Analysis

Since a response spectrum analysis is performed, it is necessary to apply the recommendations established in NCh2745 for dynamic analysis. This analysis is based on the results obtained from the static analysis but incorporates certain modifications that condition and limit the design of the isolated structure.

a) Maximum and Design Displacements

Based on the static analysis results, adjustments were made to incorporate the influence of superstructure flexibility, which allowed for a reduction in deformation demand within the isolation system. To calculate the maximum and design displacements in each direction, the periods of the conventional structure with the isolation level incorporated were considered:

0.64 s in the X-direction and 0.63 s in the Y-direction. Using the expressions described in Section II.3.7, the following values were obtained:

- Design displacement in X: $D'_{DX} = 23.93$ cm
- Design displacement in Y: $D'_{DY} = 23.95$ cm
- Maximum displacement in X: $D'_{MX} = 28.72$ cm
- Maximum displacement in Y: $D'_{MY} = 28.74$ cm

With these displacement limits, a verification was performed to ensure that the results from the structural analysis fell within the established ranges. To achieve this, the displacements of the center of mass of the 5th story (the level exhibiting the largest displacements in both directions) were analyzed without reductions:

- 5th Story displacement in X: $D_{X5} = 25.56$ cm
- 5th Story displacement in Y $D_{Y5} = 25.55$ cm

As observed, the displacements obtained at the 5th story comply with the established limits, confirming that the structure meets the required safety and design parameters.

b) Minimum Lateral Forces

The following modifications are applied to the minimum lateral forces:

$$V_b = 0.9 * V_{b,est} = 2299\ KN\ (14\%) \tag{V.8}$$

$$V_s = 0.8 * V_{s,est} = 1022\ KN\ (6\%) \tag{V.9}$$

c) Verification of Limits for V_b and V_s

After conducting the response spectrum analysis, the following lateral force values were obtained for the isolation system and the superstructure:

- Lateral force in the isolation system (X-direction): V_{bX} = 2420 KN (15%)
- Lateral force in the isolation system (Y-direction): V_{bY} = 2424 KN (15%)
- Lateral force in the superstructure (X-direction): V_{sX} = 1211 KN (8%)
- Lateral force in the superstructure (Y-direction): V_{sY} = 1212 KN (8%)

As observed, these values exceed both the minimum lateral forces obtained from the dynamic analysis and the minimum required by NCh433. Therefore, the structure complies with the regulatory requirements.

V.3 Isolation Design Spectrum

To obtain the design spectrum, it is necessary to scale the base spectrum generated in Section III.5.3 by applying the factor Z=1.25 and then reduce it using the factor B_D =1.67. This procedure incorporates the damping effect provided by the isolators into the spectrum before being implemented in the ETABS model. Figure V-2 presents the base isolation spectrum alongside the design spectrum, highlighting its most representative values.

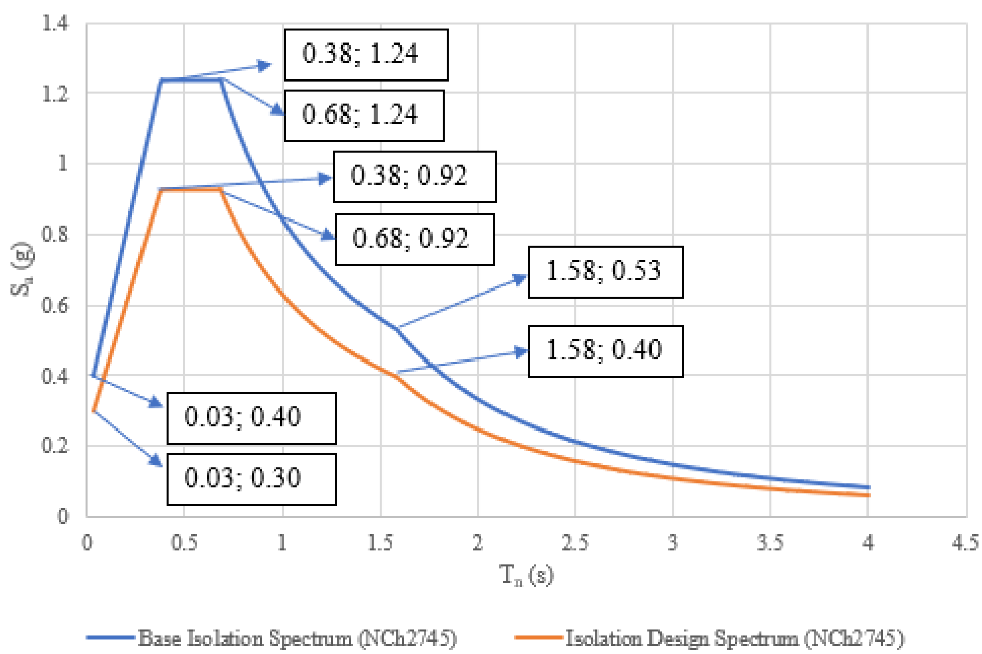


Figure V-2: Comparison between the base isolation spectrum and the design spectrum.

V.4 Results of the Response Spectrum Analysis

V.4.1 Story Shears

After performing the response spectrum analysis, it was verified that the lateral forces in both the superstructure and the isolation system remain within the established limits, as detailed in Section V.2.3 on dynamic analysis. Table V-4, Figure V-3 and Figure V-4 present the story shear forces obtained in the X and Y directions for the superstructure, which allow for an evaluation of the distribution of lateral forces along the height of the structure.

Table V-3: Story shear forces for the isolated structure.

Story	Qx (KN)	Qy (KN)
5	124	124
4	355	355
3	579	579
2	796	796
1	1008	1009
0	1210	1212

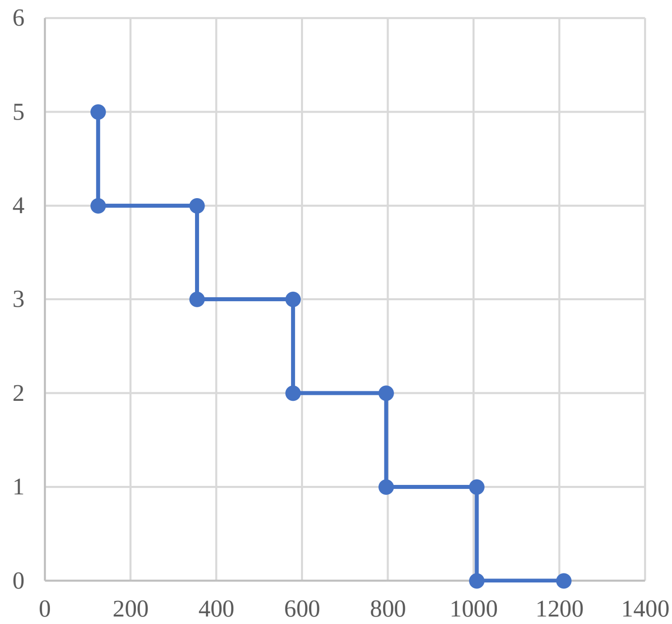


Figure V-3: Story shear forces in X direction for the isolated structure.

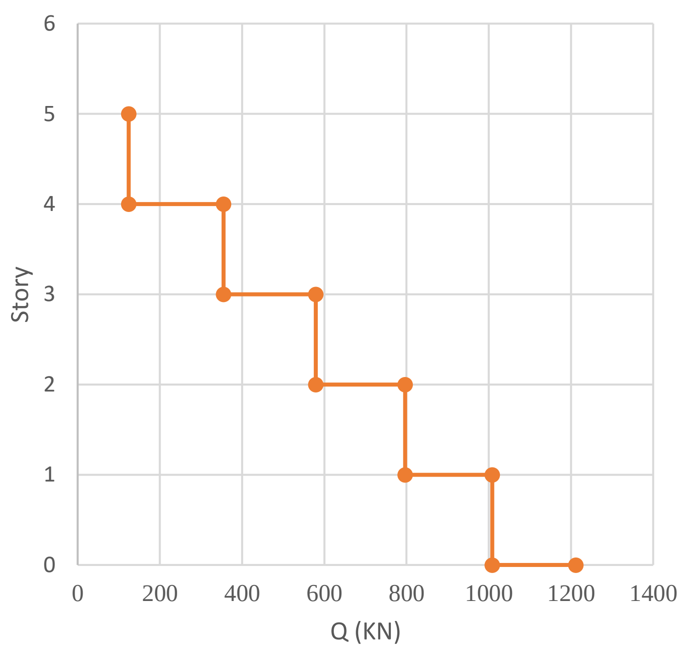


Figure V-4: Story shear forces in Y direction for the isolated structure.

V.4.2 Interstory Drifts

Regarding interstory drifts, the design was developed in compliance with the limits established by NCh2745, which specifies that drifts for a response spectrum analysis must be less than 0.0025h. It is important to note that, in this particular case, the results for the superstructure do not include the drift corresponding to the isolation level. Table V-4, Figure V-5, and Figure V-6 show the story drifts at the center of mass, the difference between the maximum drift and the center of mass drift, along with the verification of the established limits.

Table V-4: Verification of interstory drifts for the isolated structure.

Story	Diaphragm Center of Mass Drifts					Diaphragm Center of Mass Drifts - Maximum Story Drifts				
	Drift Limit (NCh2745)	CM Drift (EX)	Verification	CM Drift (EY)	Verification	Drift Limit (NCh2745)	CM Drift - Max Drift (EX)	Verification	CM Drift - Max Drift (EY)	Verification
0	0.0025	0	OK	0	OK	0.001	0	OK	0	OK
1	0.0025	0.0011	OK	0.0011	OK	0.001	0.0001	OK	0.0002	OK
2	0.0025	0.0010	OK	0.0010	OK	0.001	0.0001	OK	0.0001	OK
3	0.0025	0.0007	OK	0.0007	OK	0.001	0.0001	OK	0.0001	OK
4	0.0025	0.0005	OK	0.0005	OK	0.001	0.0000	OK	0.0001	OK
5	0.0025	0.0002	OK	0.0002	OK	0.001	0.0000	OK	0.0000	OK

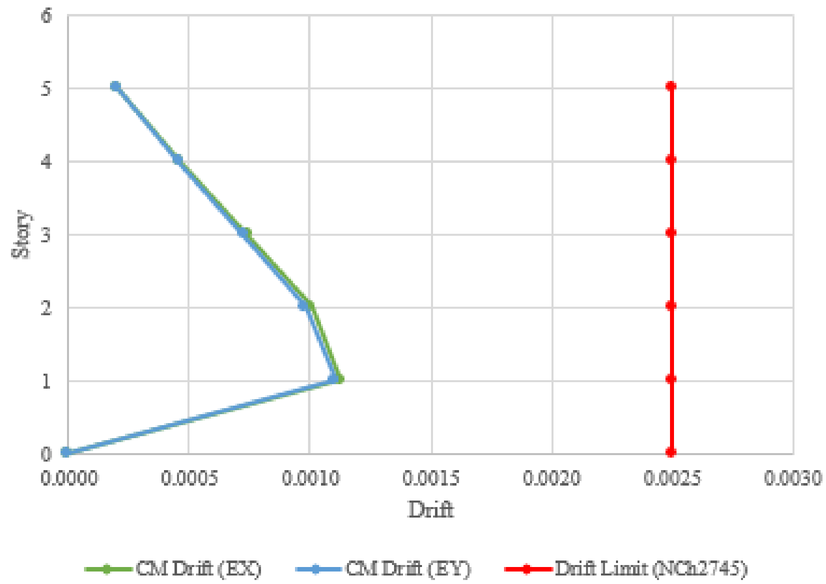


Figure V-5: Interstory drifts at the center of mass in the two analysis directions (X and Y) for the isolated structure.

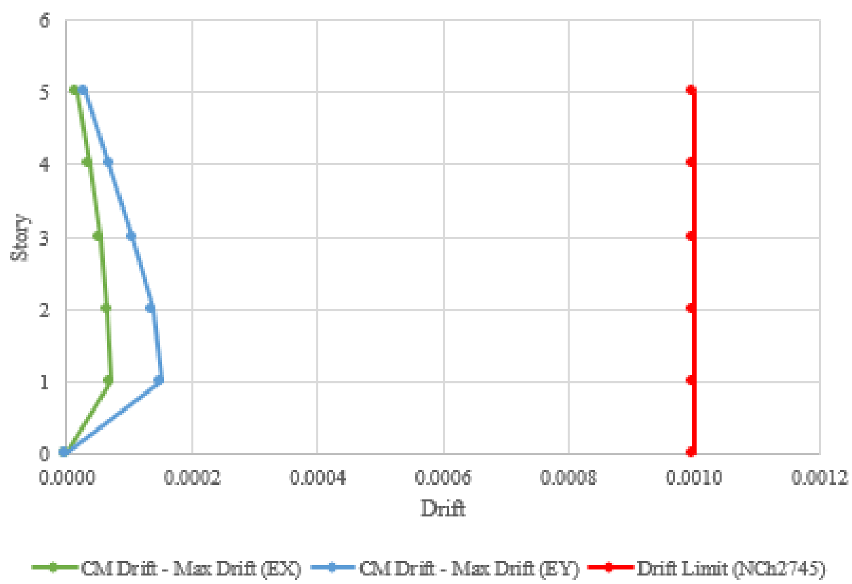


Figure V-6: Difference between the maximum drift and the drift at the center of mass in the two analysis directions (X and Y) for the isolated structure.

V.5 Reinforced Concrete Element Design

According to the procedure described in Section III-6, the design of structural elements was carried out using the previously defined final sections. The reinforcement plans and construction details are presented in Annex C. Table V-5 summarizes the final material quantities, detailing the concrete volume, the amount of reinforcing steel, and the volumetric reinforcement ratios for each structural element.

Table V-5: Material quantities and volumetric reinforcement ratios for the isolated structure.

Structural Element	**Concrete (m^3)**	**Reinforcing Steel (Kg)**	**Volumetric Reinforcement Ratio, ρ_v (Kg/m^3)**
Columns	48.00	14620	304.58
Beams	121.56	15552	127.94
Slabs	170.47	15015	88.08
Σ	**340.03**	**45187**	

Table V-6 summarizes the design results for the two-way slabs across all stories. Table V-7 details the design outcomes for the beams, considering both bending and shear, with four types of beams defined for the structure. Finally, Table V-8 compiles the design results for the columns, including flexural-compression, shear resistance, and the verification of the strong column-weak beam criterion, with one column type used throughout the structure.

Table V-6: Slab design results for the isolated structure.

Story	b (cm)	h (cm)	d (cm)	ρ_{min} (%)	ρ_{max} (%)	Maximum Moments per Story, Mu (kgf-m/m)				Reinforcement Quantity, As (cm^2/m)				Flexural Reinforcement Ratio, ρ (%)				Reinforcement			
						Mu_x (+)	Mu_x (-)	Mu_y (+)	Mu_y (-)	As_x (+)	As_x (-)	As_y (+)	As_y (-)	ρ_x (+)	ρ_x (-)	ρ_y (+)	ρ_y (-)	As_x (+)	As_x (-)	As_y (+)	As_y (-)
0	100	12	9.60	0.04	2	750	1350	740	1200	2.16	3.80	2.16	3.36	0.23	0.40	0.23	0.35	Φ8c/23	Φ8c/13	Φ8c/23	Φ8c/15
1	100	12	9.60	0.04	2	750	1350	740	1200	2.16	3.80	2.16	3.36	0.23	0.40	0.23	0.35	Φ8c/23	Φ8c/13	Φ8c/23	Φ8c/15
2	100	12	9.60	0.04	2	748	1342	738	1192	2.16	3.77	2.16	3.34	0.23	0.39	0.23	0.35	Φ8c/23	Φ8c/13	Φ8c/23	Φ8c/15
3	100	12	9.60	0.04	2	755	1315	746	1191	2.16	3.71	2.16	3.34	0.23	0.39	0.23	0.35	Φ8c/23	Φ8c/13	Φ8c/23	Φ8c/15
4	100	12	9.60	0.04	2	765	1260	740	1060	2.16	3.53	2.16	2.95	0.23	0.37	0.23	0.31	Φ8c/23	Φ8c/14	Φ8c/23	Φ8c/17
5	100	12	9.60	0.04	2	498	805	496	730	2.16	2.23	2.16	2.16	0.23	0.23	0.23	0.23	Φ8c/23	Φ8c/22	Φ8c/23	Φ8c/23

Table V-7: Beam design results for the isolated structure.

						Flexural Design										Shear Design			
Stories	Axes	Spans	b (cm)	h (cm)	d (cm)	ρ_{min} (%)	ρ_{max} (%)	Maximum Bending Moments, Mu (KN*m)		Reinforcement Quantity, As (cm^2)		Flexural Reinforcement Ratio, ρ (%)		Longitudinal Reinforcement		Maximum Shear Force (KN)	Reinforcement Quantity, Av (cm^2)	Transverse Reinforcement	
								Mu (-)	Mu (+)	As (-)	As (+)	ρ (-)	ρ (+)	As (-)	As (+)	Vu	Av	Unconfined Zone	Confined Zone
0,1,2	A, B, C, D, E	1 to 2	30	50	45.10	0.03	1.61	161.81	91.20	12.70	7.62	0.94	0.56	5φ18	3φ18	165.73	1.57	φ10@14	φ10@10
		2 to 3	30	50	45.10	0.03	1.61	149.06	114.74	10.16	10.16	0.75	0.75	4φ18	4φ18	173.58	1.57	φ10@10	φ10@10
		3 to 4	30	50	45.10	0.03	1.61	161.81	91.20	12.70	7.62	0.94	0.56	5φ18	3φ18	165.73	1.57	φ10@14	φ10@10
	1, 2, 3, 4	A to B	30	50	45.10	0.03	1.61	159.85	79.92	10.16	5.08	0.75	0.38	4φ18	2φ18	141.22	1.57	φ10@16	φ10@10
		B to C	30	50	45.10	0.03	1.61	146.12	73.06	10.16	5.08	0.75	0.38	4φ18	2φ18	138.27	1.57	φ10@16	φ10@10
		C to D	30	50	45.10	0.03	1.61	147.10	73.55	10.16	5.08	0.75	0.38	4φ18	2φ18	138.27	1.57	φ10@16	φ10@10
		D to E	30	50	45.10	0.03	1.61	159.85	79.43	10.16	5.08	0.75	0.38	4φ18	2φ18	140.23	1.57	φ10@16	φ10@10
3,4,5	A, B, C, D, E	1 to 2	30	50	45.10	0.03	1.61	108.85	54.43	7.62	5.08	0.56	0.38	3φ18	2φ18	122.58	1.57	φ10@18	φ10@10
		2 to 3	30	50	45.10	0.03	1.61	88.26	53.94	7.62	5.08	0.56	0.38	3φ18	2φ18	125.52	1.57	φ10@18	φ10@10
		3 to 4	30	50	45.10	0.03	1.61	108.85	54.43	7.62	5.08	0.56	0.38	3φ18	2φ18	125.52	1.57	φ10@18	φ10@10
	1, 2, 3, 4	A to B	30	50	45.10	0.03	1.61	110.81	55.41	7.62	5.08	0.56	0.38	3φ18	2φ18	128.47	1.57	φ10@17	φ10@10
		B to C	30	50	45.10	0.03	1.61	105.91	52.96	7.62	5.08	0.56	0.38	3φ18	2φ18	127.49	1.57	φ10@17	φ10@10
		C to D	30	50	45.10	0.03	1.61	105.91	52.96	7.62	5.08	0.56	0.38	3φ18	2φ18	127.49	1.57	φ10@17	φ10@10
		D to E	30	50	45.10	0.03	1.61	110.81	55.41	7.62	5.08	0.56	0.38	3φ18	2φ18	131.41	1.57	φ10@17	φ10@10

Table V-8: Column design results for the isolated structure.

Stories	Column ID	b (cm)	h (cm)	ρ_{min} (%)	ρ_{max} (%)	Design for flexural compression					Shear Design				Strong Column – Weak Beam
						Maximum Axial Force (KN)	Maximum Bending Moment (KN*m)	Reinforcement Quantity (cm^2)	Flexural Reinforcement Ratio	Longitudinal Reinforcement	Maximum Shear Force (KN)	Reinforcement Quantity (cm^2)	Transverse Reinforcement		
						Pu	Mu	As	ρ (%)	As	Vu	Av	Unconfined Zone	Confined Zone	$\sum M_{nc} \geq 1.2 \sum M_{nb}$
1,2,3,4,5	1A, 2A, 3A, 4A, 1B, 2B, 3B, 4B, 1C, 2C, 3C, 4C, 1D, 2D, 3D, 4D, 1E, 2E, 3E, 4E	40	40	1	6	1234.65	133.37	30.54	1.91	12φ18	251.05	3.14	φ10@10	φ10@10	OK

Figure V-7 presents the interaction diagram for the flexural-compression design of the adopted column type used throughout the isolated structure. This graph demonstrates that the column's load-bearing capacity meets the demand requirements, thereby validating its compliance with the established structural criteria.

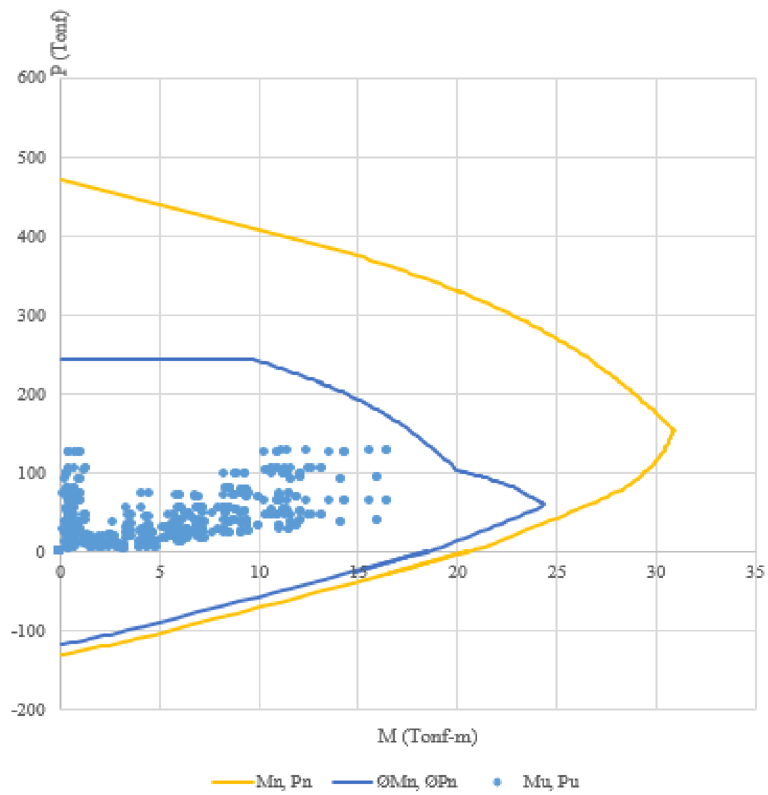


Figure V-7: Interaction diagram and design loads for the flexural compression design of the 40 cm x 40 cm column section with 12φ18 longitudinal reinforcement for the isolated structure.

V.6 Comparative Analysis

This section presents a comparative analysis of the results obtained for both the conventional and isolated structures. The objective is to evaluate the differences in structural behavior and the design of the reinforced concrete elements. Table V-9 summarizes the key results and the percentage variation of the isolated structure compared to the conventional structure.

Table V-9: Comparative results of the conventional and isolated structures.

Parameters		**Conventional Structure**		**Isolated Structure**	**% Variation of Isolated Structure Compared to Conventional Structure**	
		Chilean Spectrum	**Turkish Spectrum**		**Chilean Spectrum**	**Turkish Spectrum**
Slabs	h(m)	0.12	0.12	0.12	0%	0%
Beams	b(m)	0.45	0.30	0.30	-33%	0%
	h (m)	0.60	0.50	0.50	-17%	0%
Columns	b(m)	0.55	0.40	0.40	-27%	0%
	h (m)	0.55	0.40	0.40	-27%	0%
Structure Weight	W (KN)	16214	13331	16059	-1%	20%
Fundamental Periods	T_x (s)	0.40	0.64	2.57	536%	303%
	T_y (s)	0.40	0.63	2.57	544%	307%
Design pseudo-acceleration	Sa_x (g)	0.31	0.13	0.15	-51%	16%
Base Shear	Q_x (KN)	2724	1496	1210	-56%	-19%
	Q_y (KN)	2724	1518	1212	-56%	-20%
Flexural Reinforcement Ratio (Max)	ρ_{slab} (%)	0.40	0.40	0.40	0%	0%
	ρ_{beam} (%)	0.78	1.13	0.94	21%	-17%
	ρ_{col} (%)	2.01	2.85	1.91	-5%	-33%
Concrete Volume	V_c (m^3)	399	291	340	-15%	17%
Reinforcing Steel Quantity	W_s (Kg)	61907	44416	45187	-27%	2%
Volumetric Reinforcement Ratio	$\rho_{v,\,slab}$ (Kg/m^3)	97	87	88	-9%	1%
	$\rho_{v,\,beam}$ (Kg/m^3)	142	158	128	-10%	-19%
	$\rho_{v,\,col}$ (Kg/m^3)	264	334	305	15%	-9%

Figure V-8 presents a comparison of the Chilean, Turkish, and isolation design spectra. Additionally, each spectrum highlights the period corresponding to the first vibration mode of each structure, along with the associated pseudo-acceleration.

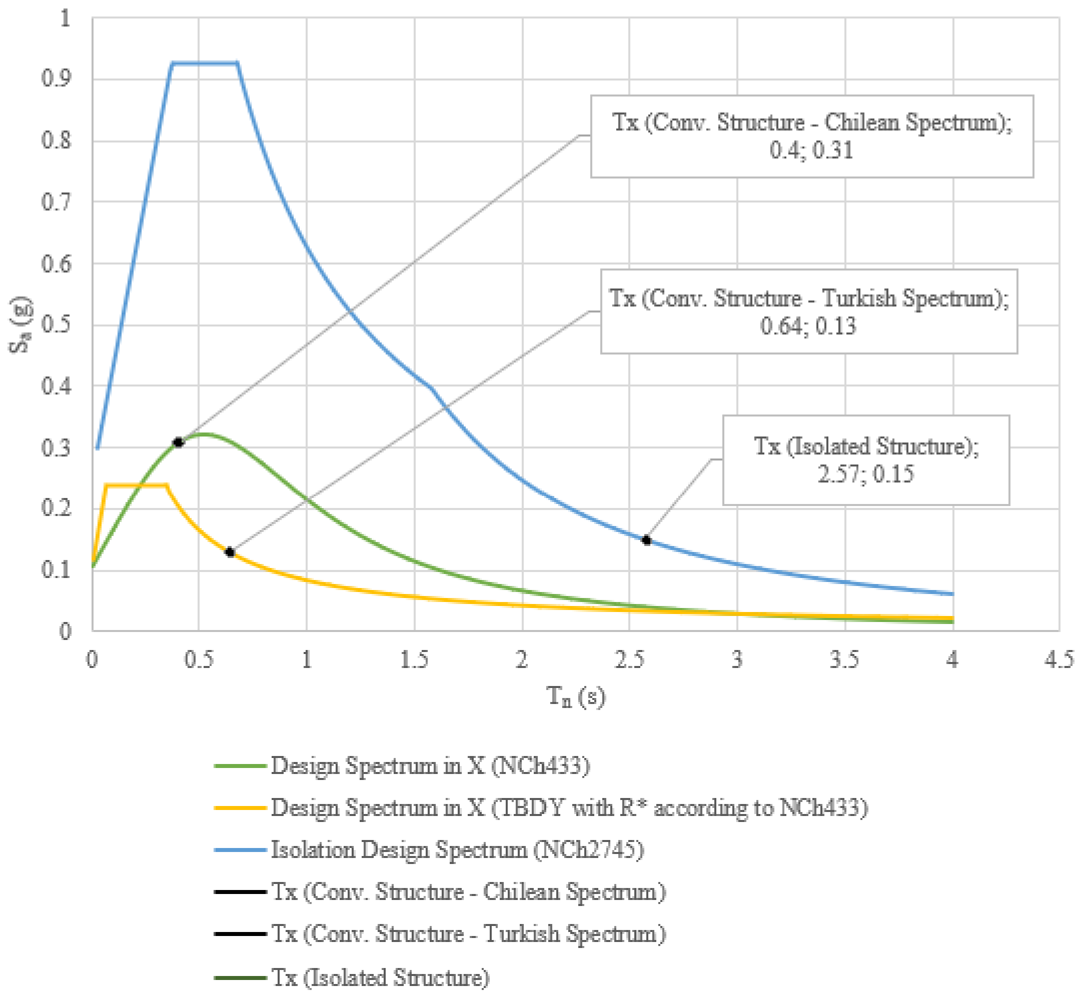


Figure V-8: Comparison of the Chilean, Turkish, and isolation design spectra, including the period and pseudo-acceleration corresponding to the first vibration mode of each structure.

The results indicate that, in general terms, the isolated structure experiences lower demands compared to the conventional structure. However, when analyzing the final geometry of the structural elements, it is observed that the dimensions obtained for the conventional structure with the Turkish spectrum and the isolated structure are equivalent. In contrast, with the Chilean spectrum, reductions in beam and column dimensions were recorded, ranging from 17% to 33%.

Regarding the pseudo-acceleration of the first mode, a value of 0.15g was obtained, approximately half of that corresponding to the conventional structure with the Chilean spectrum (0.31g) and similar to that of the Turkish spectrum (0.13g). In terms of the total structural weight, no significant variations were observed compared to the conventional structure with the Chilean spectrum, as the inclusion of the slab and additional beams at the base compensates for the differences. However, in the case of the Turkish spectrum, the isolated structure presents a 20% higher weight, since both share the same structural geometry, and in this case, the difference is due to the additional slab and beams.

Regarding fundamental periods, the isolated structure reaches a value of 2.57 s, representing an increase of more than five times compared to the conventional structure with the Chilean spectrum (0.40 s) and approximately three times compared to the Turkish spectrum (0.64 s). This increase is attributed to the greater horizontal flexibility provided by the elastomeric isolators. Additionally, base shear forces are reduced by 56% and 20% compared to the conventional structure with the Chilean and Turkish spectra, respectively.

In terms of structural reinforcement, the required amount of steel decreases by up to 27% and 2% in relation to the conventional structure with the Chilean and Turkish spectra, respectively. Furthermore, a significant reduction in the volumetric reinforcement of beams is observed, around 10% and 19% compared to the conventional structure with the Chilean and Turkish spectra, respectively. Regarding longitudinal reinforcement ratios, compared to the conventional structure with the Chilean spectrum, a 21% increase in beams and a 5% reduction in columns were observed. In the case of the Turkish spectrum, reductions of 17% in beams and 33% in columns were recorded.

VI CONCLUSIONS AND RECOMMENDATIONS

VI.1 Summary and Conclusions

This study focused on a comparative analysis of the structural design of a prototype reinforced concrete residential building, examining its seismic behavior under different design spectra, both with and without base isolation. Through this evaluation, key differences between the models were identified, highlighting their impact on structural performance and design requirements in each case. The main conclusions are as follows:

Both codes share a performance-based approach, the use of elastic response spectra, and the application of capacity-based design principles to ensure ductility. However, there are significant differences in their seismic zoning: Chile is based on subduction activity, while Turkey considers active crustal faults. The TBDY (2018) provides more specificity in soil characterization and site effects, while the NCh433 (2009) imposes stricter requirements in the design of tall structures and greater demand for seismic force reduction.

An attempt was made to equate a Chilean spectrum with the Turkish spectrum to establish a point of comparison. However, due to the geographical and seismic differences between both countries, it was not possible to find an exact equivalence in terms of peak acceleration and predominant periods. As a result, the model designed using the Chilean spectrum exhibited higher structural demands compared to the model designed with the Turkish spectrum, leading to more robust structural elements and increased internal forces.

To adjust the Turkish spectrum to the Chilean regulatory framework, a reduction factor R* based on NCh433 (2009) was applied, considering soil type D conditions in a seismic zone 3. However, this procedure is not usual, as reduction factors are determined within the context of the same code. The TBDY (2018) uses different criteria for generating its spectrum, which limits the validity of a direct comparison using reduction factors from another code.

The conventional structure designed using the Chilean spectrum exhibited fundamental periods of approximately 0.40 s, compared to 0.64 s for the structure designed with the Turkish spectrum. These periods correspond to pseudo-accelerations of 0.31g and 0.13g, respectively. As a result, the structure designed with the Chilean spectrum experienced higher base shear forces and greater demands on structural elements compared to the one designed with the Turkish spectrum. It is important to note that an increase in the dimensions of the structural elements was necessary for the model with the Chilean spectrum. Maintaining the same dimensions as the Turkish spectrum model resulted in interstory drifts exceeding the limits established by NCh433 (2009). This adjustment was essential to meet regulatory requirements and, in turn, is one of the key factors explaining the differences in fundamental periods between both models.

In the second phase of the study, base isolation was incorporated into the case study building, significantly improving its seismic performance. When compared to the conventional structure designed with the Chilean spectrum, substantial reductions were observed in stress demands, seismic demand, and the size of the structural elements. This led to a decrease in story shears and an increase in structural resilience. In the case of the Turkish spectrum, reductions in base shears were also recorded, although the dimensions of the elements were equivalent to those of the isolated structure, with a notable reduction in reinforcement quantities. It is important to highlight that, based on the results from the interstory drifts of the isolated structure, further optimization of the structural element dimensions could be explored. However, the dimensions presented in this document were maintained to comply with the minimum seismic design requirements for reinforced concrete, as set by NCh430 (2008) and ACI 318-19. Overall, the isolated structure demonstrated lower interstory drifts and a more favorable seismic response compared to the conventional structure, confirming the effectiveness of seismic isolation in mitigating structural damage and improving the building's overall behavior during seismic events.

This study confirms that regulatory differences have a significant impact on structural design and that the application of the criteria from one code in the context of another can lead to inconsistencies. Furthermore, the incorporation of seismic isolation proves to be an effective strategy for improving the resilience of structures in high seismicity zones, reducing structural demands, and optimizing element design. This comparative analysis provides a foundation for future research aimed at evaluating the applicability of different seismic standards and the performance of seismic protection systems in various geographical environments.

VI.2 Recommendations for Future Research

While this study was based on a response spectrum analysis, future research could employ response history analysis or nonlinear analysis to more accurately assess the structural behavior under real seismic movements, considering effects of plasticity and material degradation.

It is recommended to conduct the reverse study of the present work, that is, applying the reduction factors of the Turkish code (TBDY, 2018) within the context of the Chilean code (NCh433, 2009), and analyzing the structure's behavior under these criteria. This would allow for the evaluation of the effectiveness and compatibility of the design approaches between the two codes.

This study used elastomeric isolators, but future research could analyze the performance of other seismic isolation systems, such as friction pendulum bearings or the incorporation of energy dissipators, assessing their impact on reducing forces and structural drifts.

It is recommended to expand the study to include other structural typologies, such as mixed steel-concrete buildings, structures with shear wall systems, or moment-resisting frame structures with a different number of stories. This would allow for an evaluation of how regulatory differences and the seismic conditions specific to each region affect the behavior and design of various building types. Such an expansion of the study would contribute to a more comprehensive understanding of the seismic performance of different structural configurations and help optimize their designs based on the applicable regulatory requirements.

Finally, it would be relevant to complement these studies with a cost-benefit analysis of seismic design strategies, evaluating the economic feasibility of different regulatory approaches and seismic protection systems to optimize structural design from both a technical and financial perspective.

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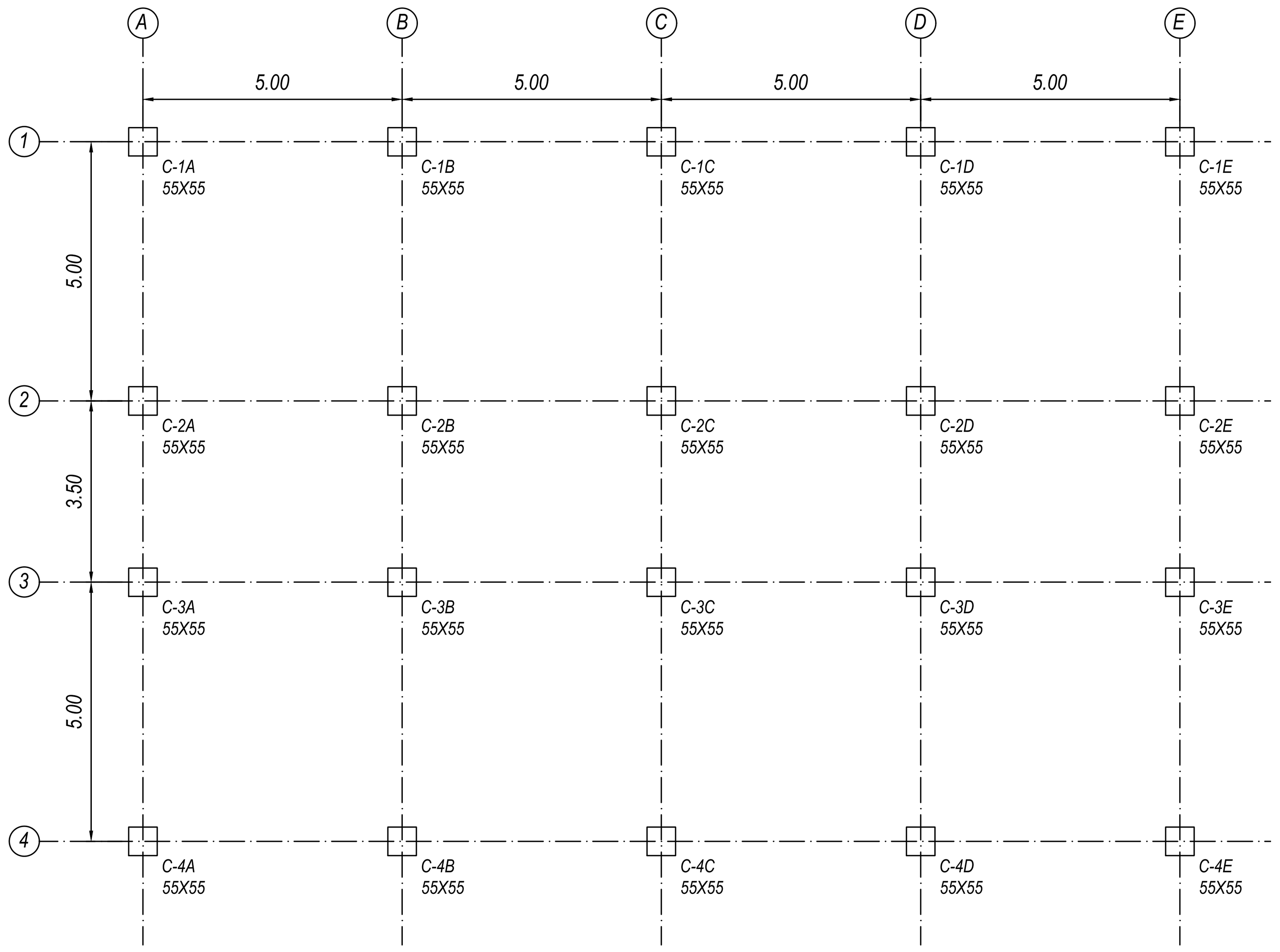
ANNEXES

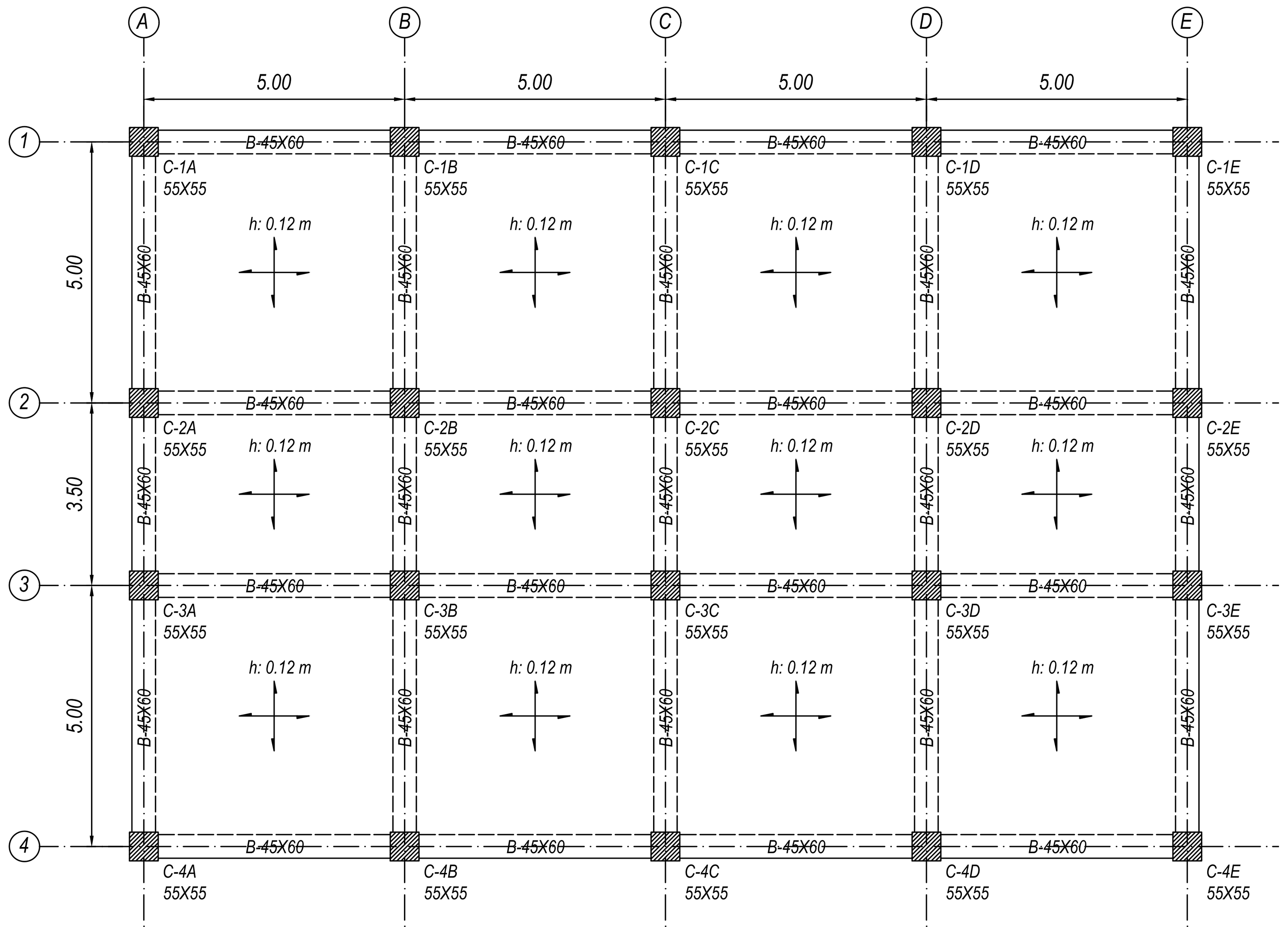
ANNEX A:

PLANS OF CONVENTIONAL STRUCTURE WITH CHILEAN SPECTRUM

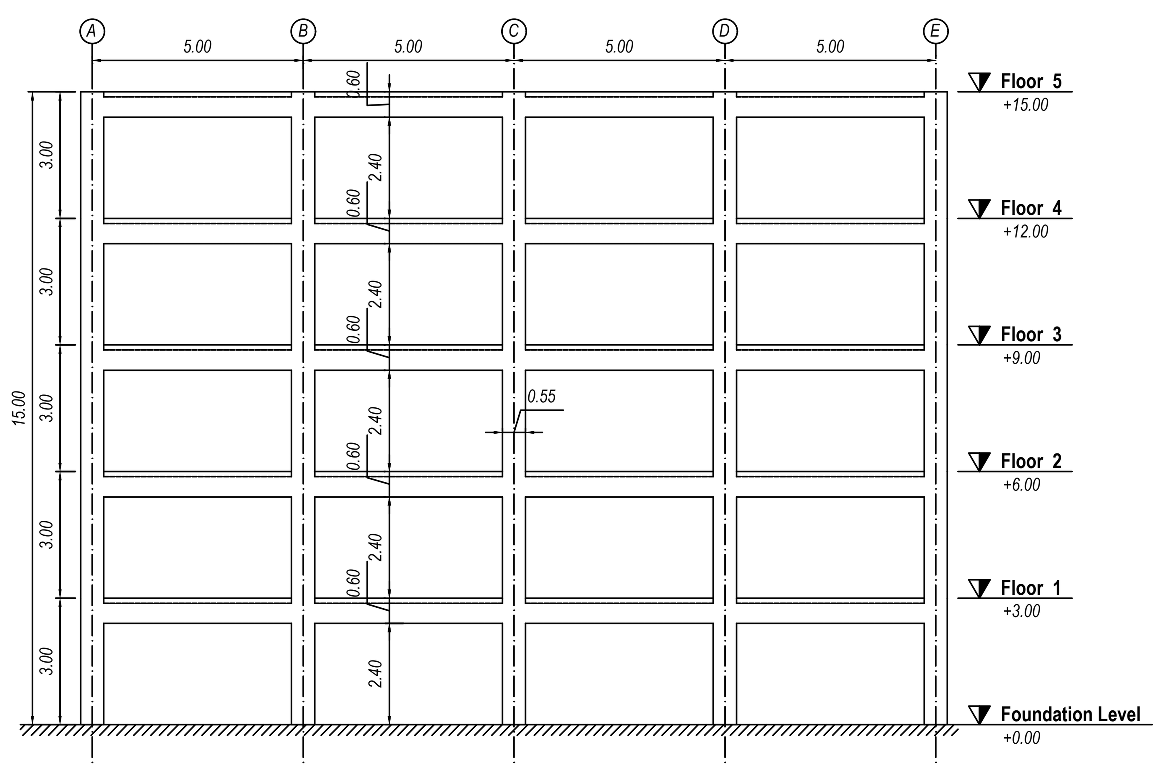
① Column Staking
Scale 1:100



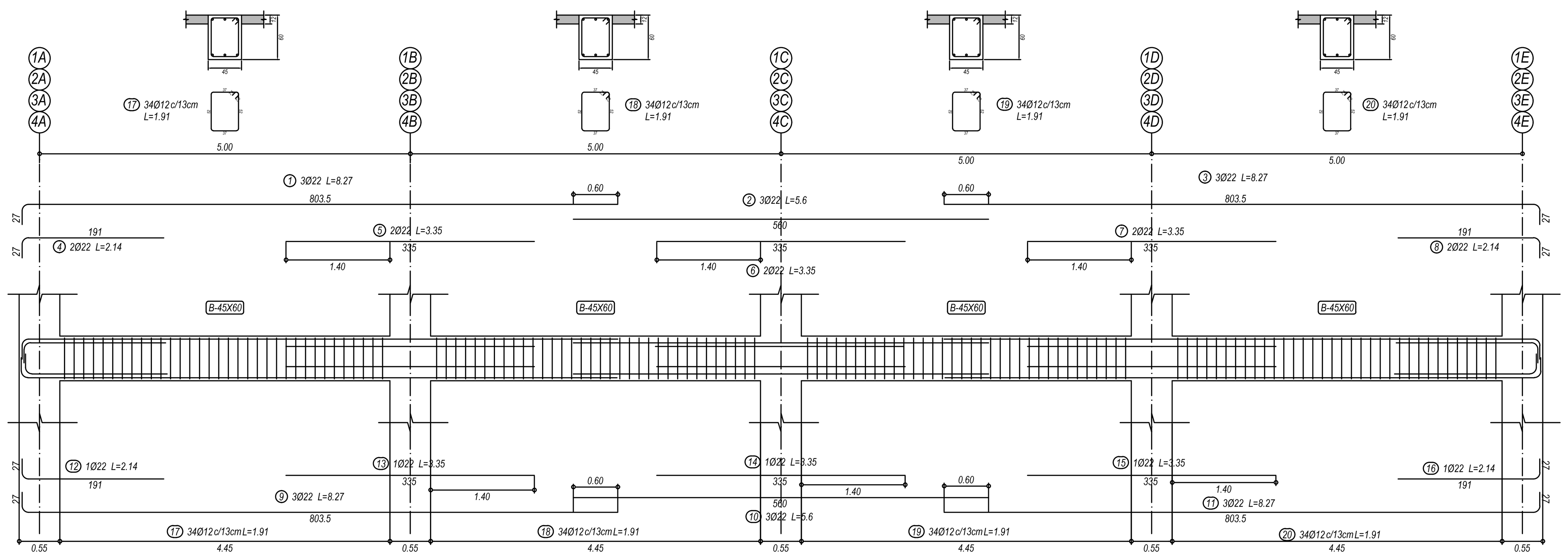
② Beams (Floor 1, Floor 2 and Floor 3)
Scale 1:100



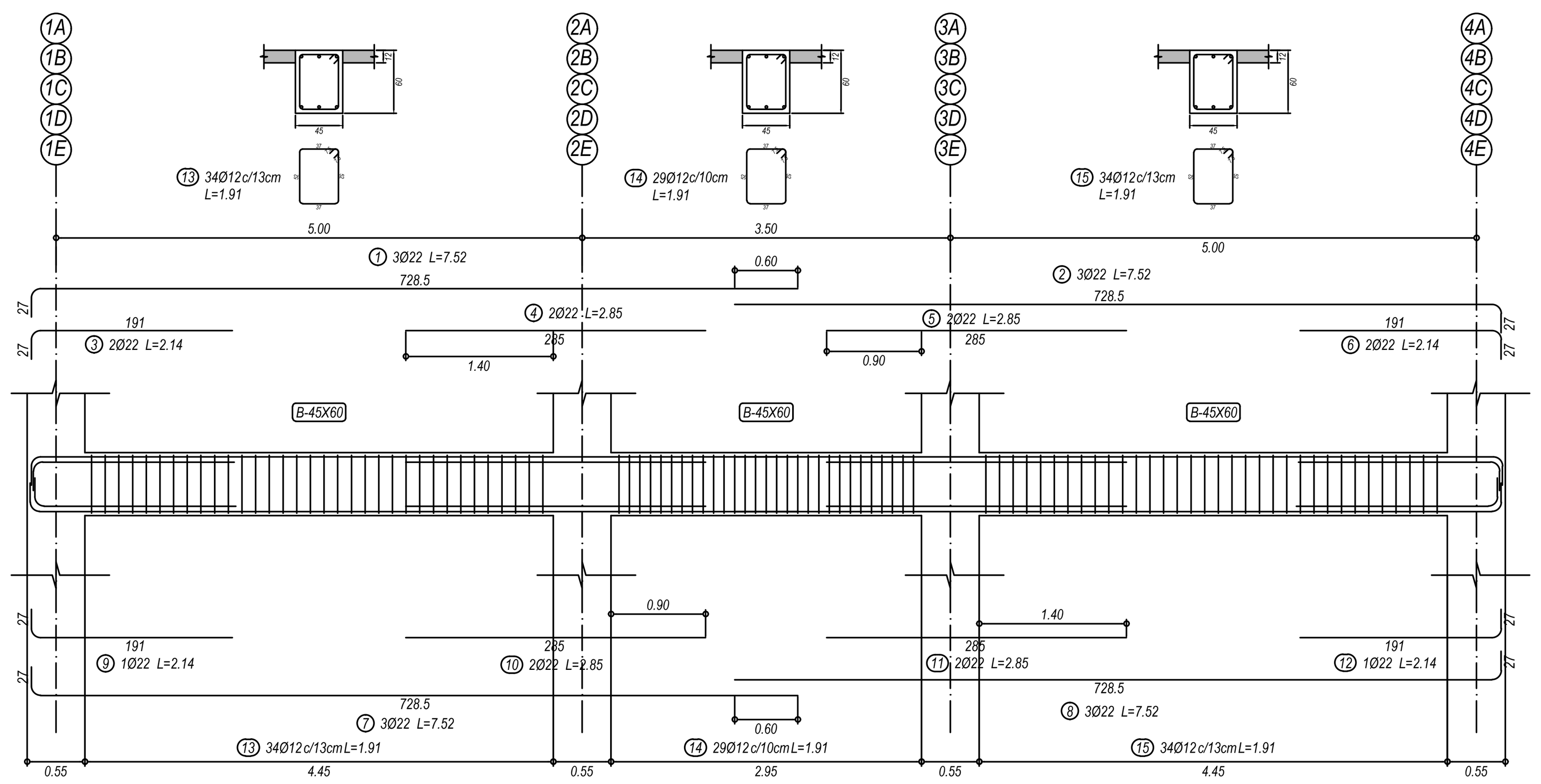
⑤ Building Elevation
Scale 1:150



③ Beam Rebar Detailing Axes 1,2,3,4 (Floor 1, Floor 2 and Floor 3)
Scale 1:60



④ Beam Rebar Detailing Axes A,B,C,D,E (Floor 1, Floor 2 and Floor 3)
Scale 1:60



⑥ Structural Scheme
Not to scale

Rebar Schedule de Beams eje A,B,C,D,E (Floor 1, Floor 2 and Floor 3)

Mark	Ø [mm]	Scheme [cm]	Length [m]	N°	Weight [kg]
1	Ø22	27 / 728.5	7.52	3	67.3
2	Ø22	27 / 728.5	7.52	3	67.3
3	Ø22	27 / 191	2.14	2	12.79
4	Ø22	285	2.85	2	17.01
5	Ø22	285	2.85	2	17.01
6	Ø22	27 / 191	2.14	2	12.79
7	Ø22	728.5 / 27	7.52	3	67.3
8	Ø22	728.5 / 27	7.52	3	67.3
9	Ø22	191 / 27	2.14	1	6.39
10	Ø22	285	2.85	2	17.01
11	Ø22	285	2.85	2	17.01
12	Ø22	191 / 27	2.14	1	6.39
13	Ø12	52 x 37, 7.5	1.91	34	57.74
14	Ø12	52 x 37, 7.5	1.91	29	49.25
15	Ø12	52 x 37, 7.5	1.91	34	57.74

Total Weight = 540 kg

Rebar Schedule de Beams eje 1,2,3,4 (Floor 1, Floor 2 and Floor 3)

Mark	Ø [mm]	Scheme [cm]	Length [m]	N°	Weight [kg]
1	Ø22	27 / 803.5	8.27	3	74.01
2	Ø22	560	5.6	3	50.13
3	Ø22	27 / 803.5	8.27	3	74.01
4	Ø22	27 / 191	2.14	2	12.79
5	Ø22	335	3.35	2	19.99
6	Ø22	335	3.35	2	19.99
7	Ø22	335	3.35	2	19.99
8	Ø22	27 / 191	2.14	2	12.79
9	Ø22	27 / 803.5	8.27	3	74.01
10	Ø22	560	5.6	3	50.13
11	Ø22	27 / 803.5	8.27	3	74.01
12	Ø22	191 / 27	2.14	1	6.39
13	Ø22	335	3.35	1	10
14	Ø22	335	3.35	1	10
15	Ø22	335	3.35	1	10
16	Ø22	27 / 191	2.14	1	6.39
17	Ø12	52 x 37, 7.5	1.91	34	57.74
18	Ø12	52 x 37, 7.5	1.91	34	57.74
19	Ø12	52 x 37, 7.5	1.91	34	57.74
20	Ø12	52 x 37, 7.5	1.91	34	57.74

Total Weight = 756 kg

CONVENTIONAL STRUCTURE DESIGNED FOR CHILEAN SEISMIC SPECTRUM

Structural system: *Three-dimensional reinforced concrete moment-resisting frame*

Design codes: *NCh433.Of1996-Mod. 2009, NCh430.Of2008*

Material properties:

Concrete: G25

Reinforcing steel: A630-420H

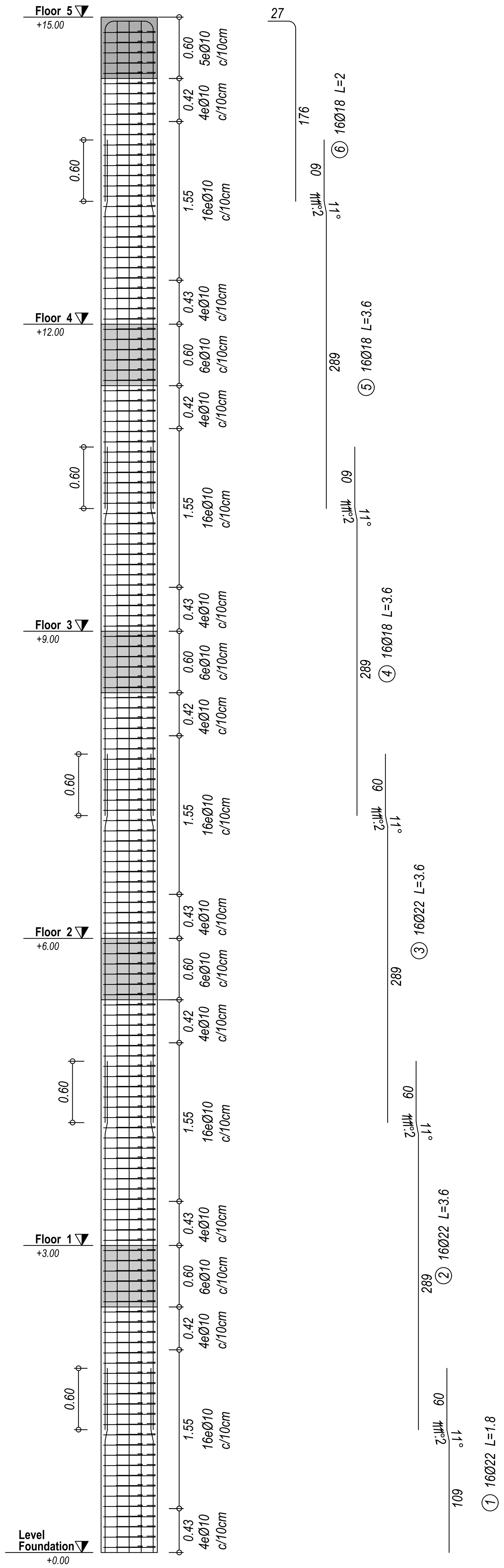
Material quantities:

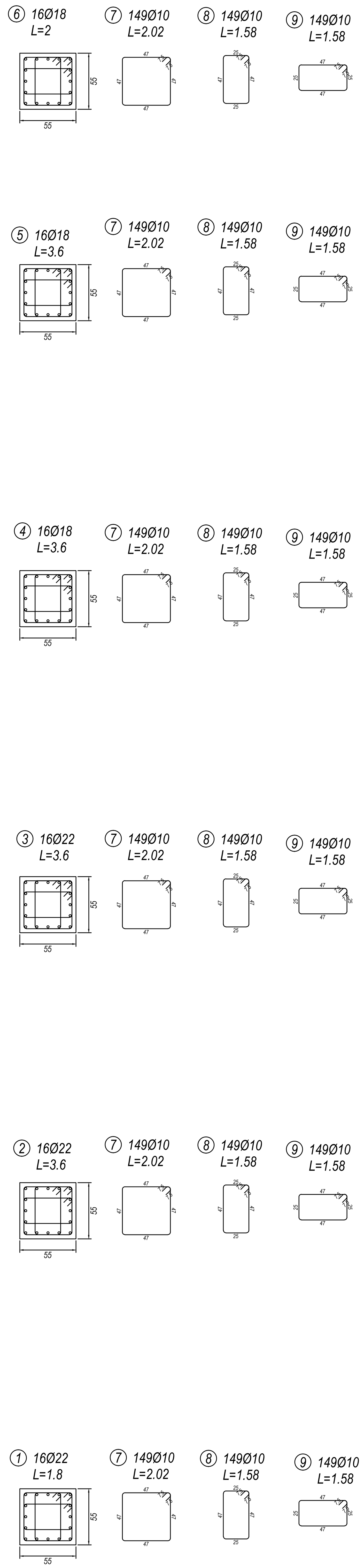
Structural element	Concrete (m^3)	Reinforcing steel (kg)	Reinforcement ratio (kg/m^3)
Columns	90.75	23980.00	264.24
Beams	176.11	25030.00	142.13
Slabs	132.61	12897.00	97.26
Total	**399.47**	**61907.00**	

Designed by: *Sergio Tito Cardozo Nava*

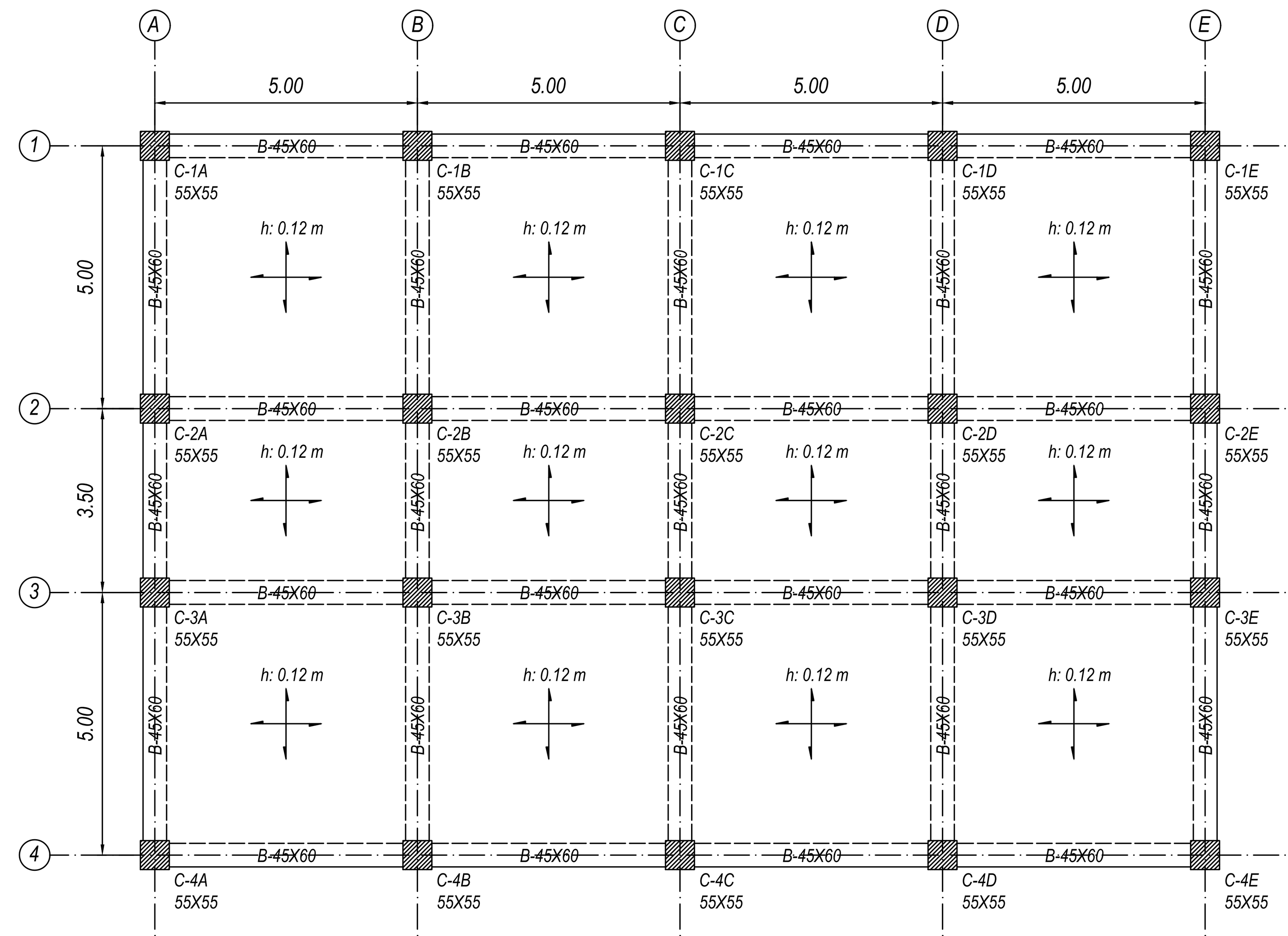
Sheet No.: 1 of 3 **Date:** March 2025

② Column Rebar Detailing
Scale 1:30

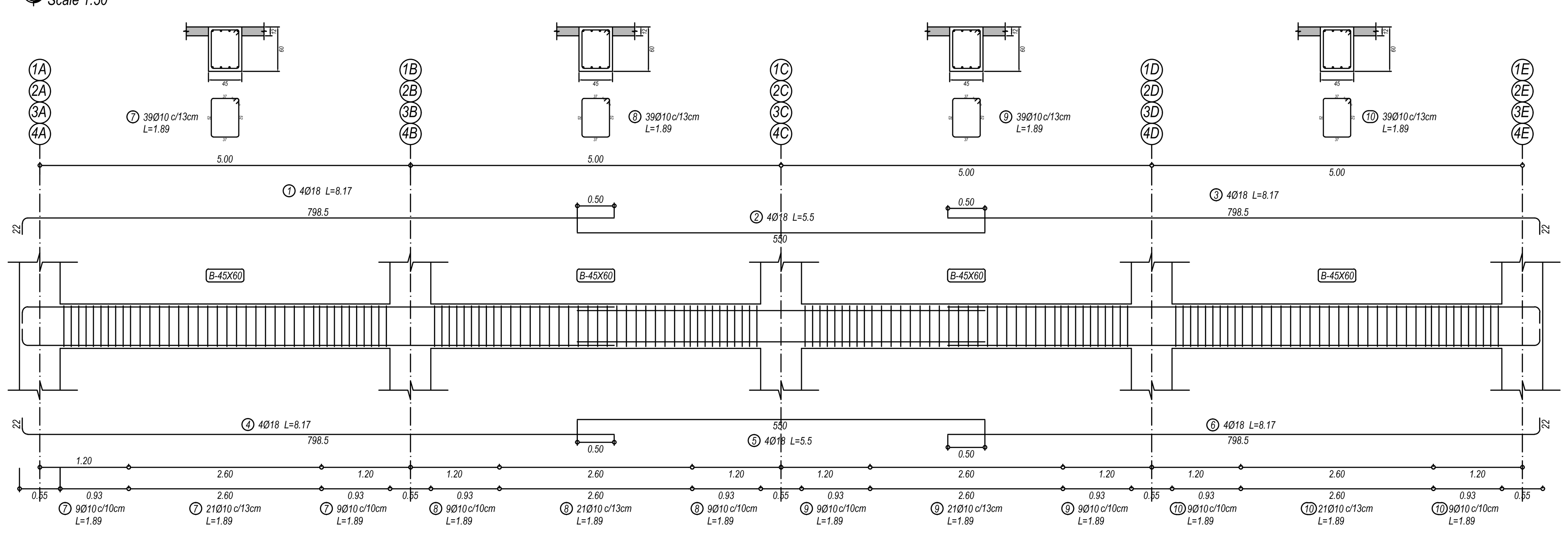




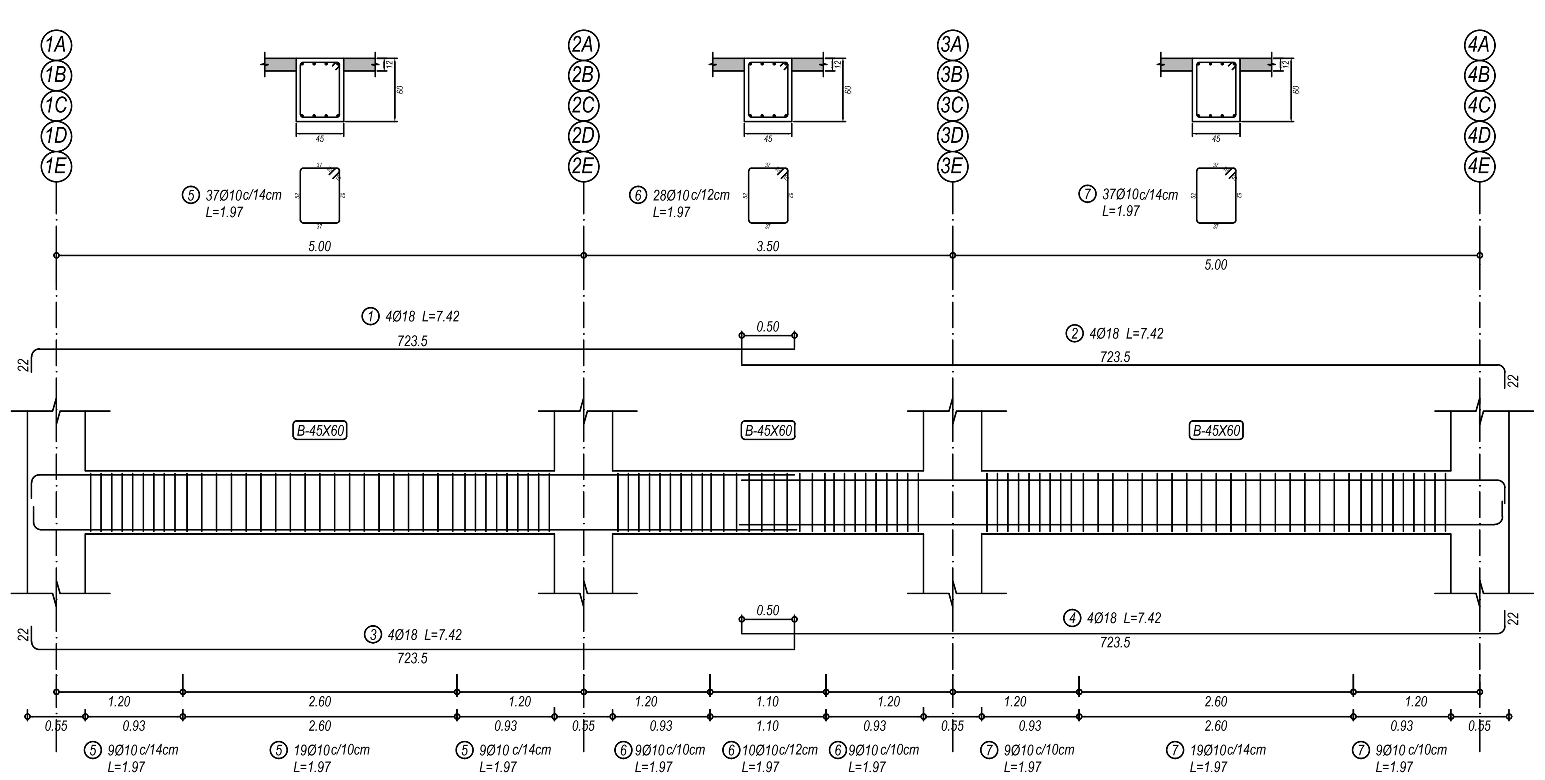
① Beams (Floor 4 and Floor 5)
Scale 1:100



① Beam Rebar Detailing Axes 1,2,3,4 (Floor 4 and Floor 5)
Scale 1:50



① Beam Rebar Detailing Axes A,B,C,D,E (Floor 4 and Floor 5)
Scale 1:50



Rebar Schedule de Beams A,B,C,D,E (Floor 4 and Floor 5)

Mark	Ø [mm]	Scheme [cm]	Length [m]	N°	Weight [kg]
①	Ø18	22 723.5	7.42	4	59.32
②	Ø18	22 723.5	7.42	4	59.32
③	Ø18	723.5 22	7.42	4	59.32
④	Ø18	723.5 22	7.42	4	59.32
⑤	Ø10	52 10 10 37 37 52	1.97	37	44.84
⑥	Ø10	52 10 10 37 37 52	1.97	28	33.93
⑦	Ø10	52 10 10 37 37 52	1.97	37	44.84
Total Weight = 361 kg					

Rebar Schedule de Beams 1,2,3,4 (Floor 4 and Floor 5)

Mark	Ø [mm]	Scheme [cm]	Length [m]	N°	Weight [kg]
①	Ø18	22 798.5	8.17	4	65.31
②	Ø18	550	5.5	4	43.95
③	Ø18	22 798.5	8.17	4	65.31
④	Ø18	798.5 22	8.17	4	65.31
⑤	Ø18	550	5.5	4	43.95
⑥	Ø18	798.5 22	8.17	4	65.31
⑦	Ø10	52 37 37 52	1.89	39	45.34
⑧	Ø10	52 37 37 52	1.89	39	45.34
⑨	Ø10	52 37 37 52	1.89	39	45.34
⑩	Ø10	52 37 37 52	1.89	39	45.34
Total Weight = 531 kg					

CONVENTIONAL STRUCTURE DESIGNED FOR CHILEAN SEISMIC SPECTRUM

Structural system: *Three-dimensional reinforced concrete moment-resisting frame*

Design codes: *NCh433.Of1996-Mod. 2009, NCh430.Of2008*

Material properties:

Concrete: *G25*

Reinforcing steel: *A630-420H*

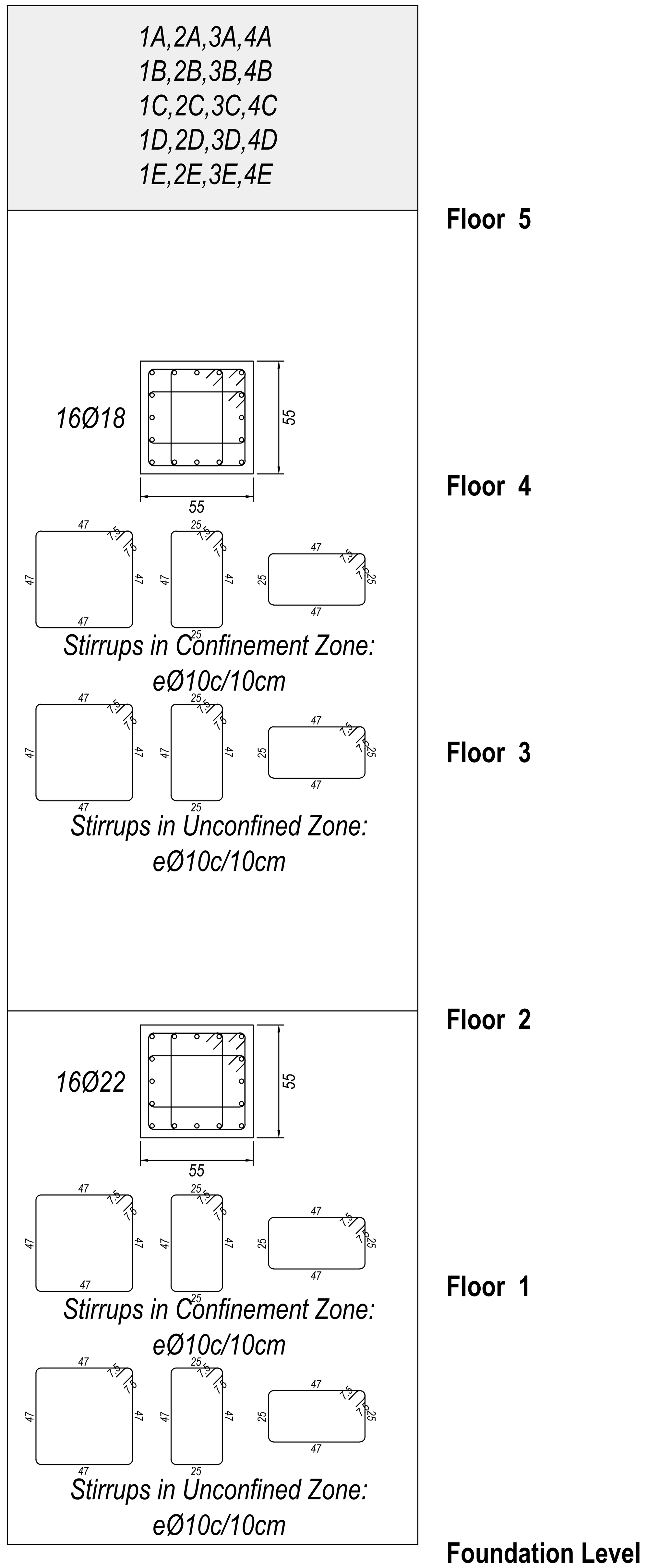
Material quantities:

Structural element	Concrete (m³)	Reinforcing steel (kg)	Reinforcement ratio (kg/m³)
Columns	90.75	23980.00	264.24
Beams	176.11	25030.00	142.13
Slabs	132.61	12897.00	97.26
Total	**399.47**	**61907.00**	

Designed by: *Sergio Tito Cardozo Nava*

Sheet No.: *2 of 3* **Date:** *March 2025*

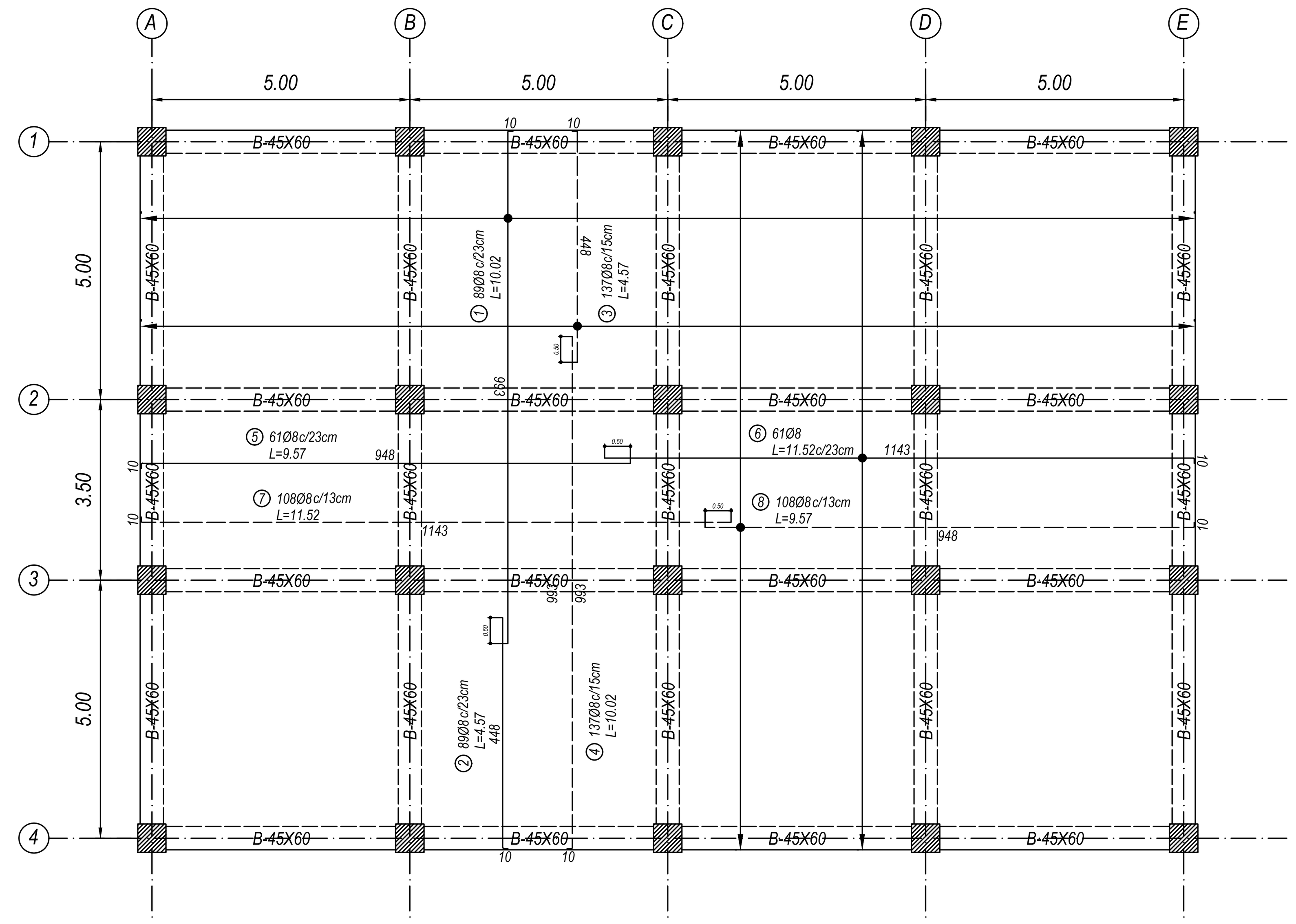
⑤ Column Schedule
Scale 1:50



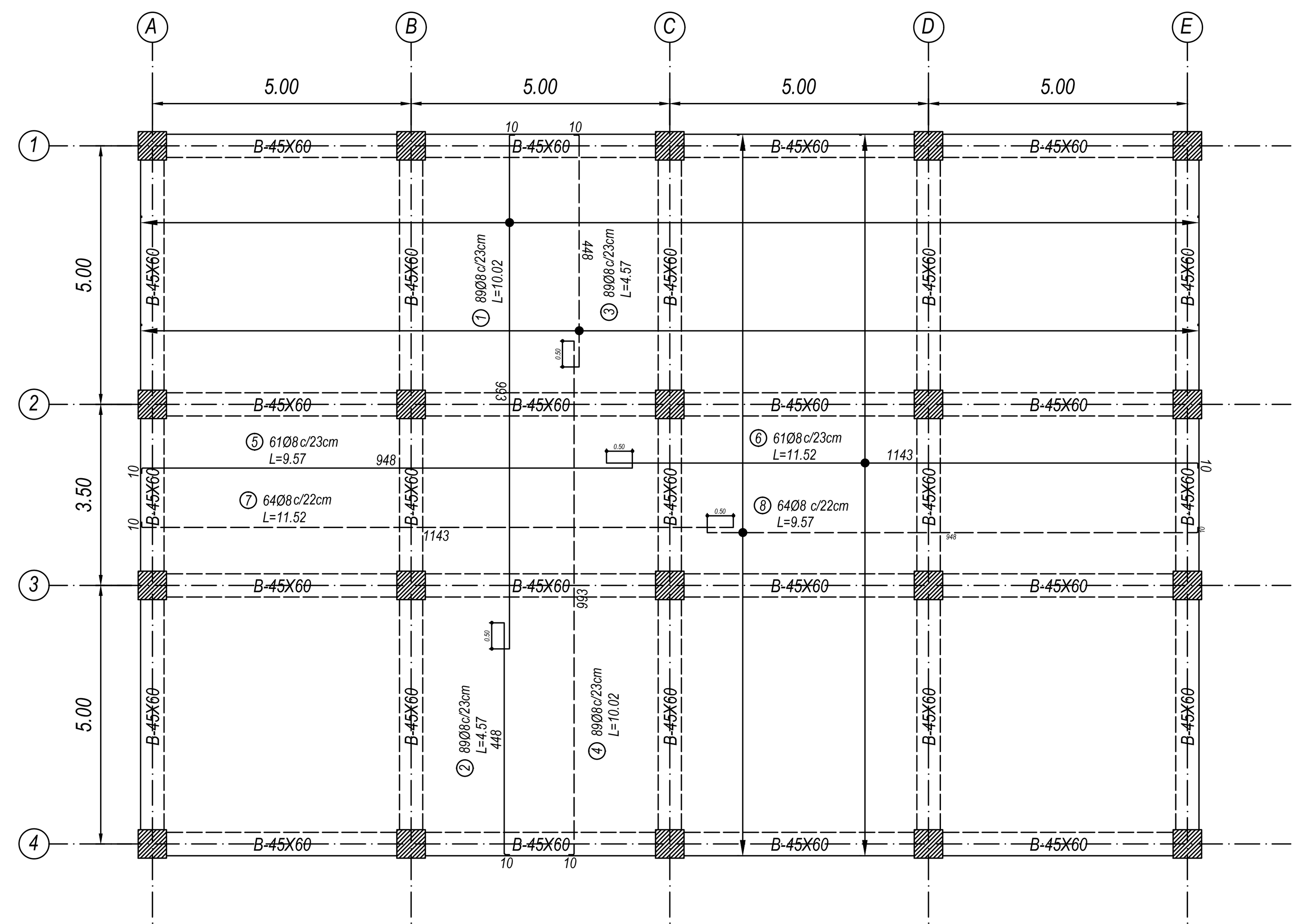
Rebar Schedule Columns

Mark	Ø [mm]	Scheme [cm]	Length [m]	N°	Weight [kg]
1	Ø22	109, 11°, 60, 111.2	1.8	16	86.04
2	Ø22	289, 11°, 60, 111.2	3.6	16	171.98
3	Ø22	289, 11°, 60, 111.2	3.6	16	171.98
4	Ø18	289, 11°, 60, 111.2	3.6	16	115.13
5	Ø18	289, 11°, 60, 111.2	3.6	16	115.13
6	Ø18	176, 27	2	16	63.89
7	Ø10	47 x 47, 7.5	2.02	149	185.16
8	Ø10	47 x 25, 7.5	1.58	149	144.74
9	Ø10	47 x 25, 7.5	1.58	149	144.74
Total Weight = 1199 kg					

① Slab Reinforcement Detail (Floor 1 a Floor 4)
Scale 1:100



② Slab Reinforcement Detail (Floor 5)
Scale 1:100



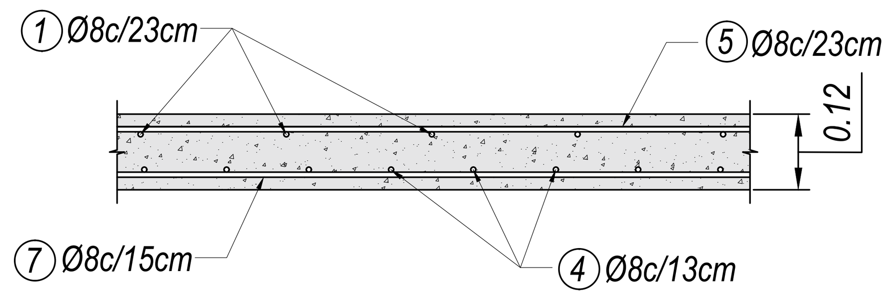
Slab Rebar Schedule (Floor 1 a Floor 4)

Mark	Ø [mm]	Scheme [cm]	Length [m]	N°	Weight [kg]
1	Ø8	10, 993	10.02	89	351.93
2	Ø8	10, 448	4.57	89	160.54
3	Ø8	448, 10	4.57	137	247.12
4	Ø8	10, 993	10.02	137	541.74
5	Ø8	10, 948	9.57	61	230.38
6	Ø8	1143, 10	11.52	61	277.32
7	Ø8	1143, 10	11.52	108	490.99
8	Ø8	948, 10	9.57	108	407.89
Weight Total = 2708 kg					

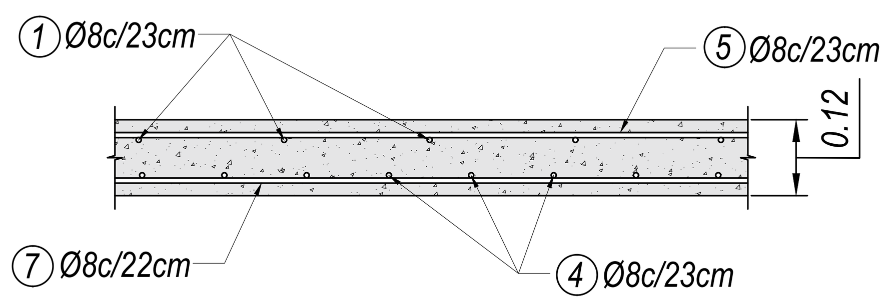
Slab Rebar Schedule (Floor 5)

Mark	Ø [mm]	Scheme [cm]	Length [m]	N°	Weight [kg]
1	Ø8	10, 993	10.02	89	351.93
2	Ø8	10, 448	4.57	89	160.54
3	Ø8	448, 10	4.57	89	160.54
4	Ø8	10, 993	10.02	89	351.93
5	Ø8	10, 948	9.57	61	230.38
6	Ø8	1143, 10	11.52	61	277.32
7	Ø8	1143, 10	11.52	64	290.95
8	Ø8	948, 10	9.57	64	241.71
Weight Total = 2065 kg					

③ Slab Rebar Detailing (Floor 1 a Floor 4)
Scale 1:10



④ Slab Rebar Detailing (Floor 5)
Scale 1:10



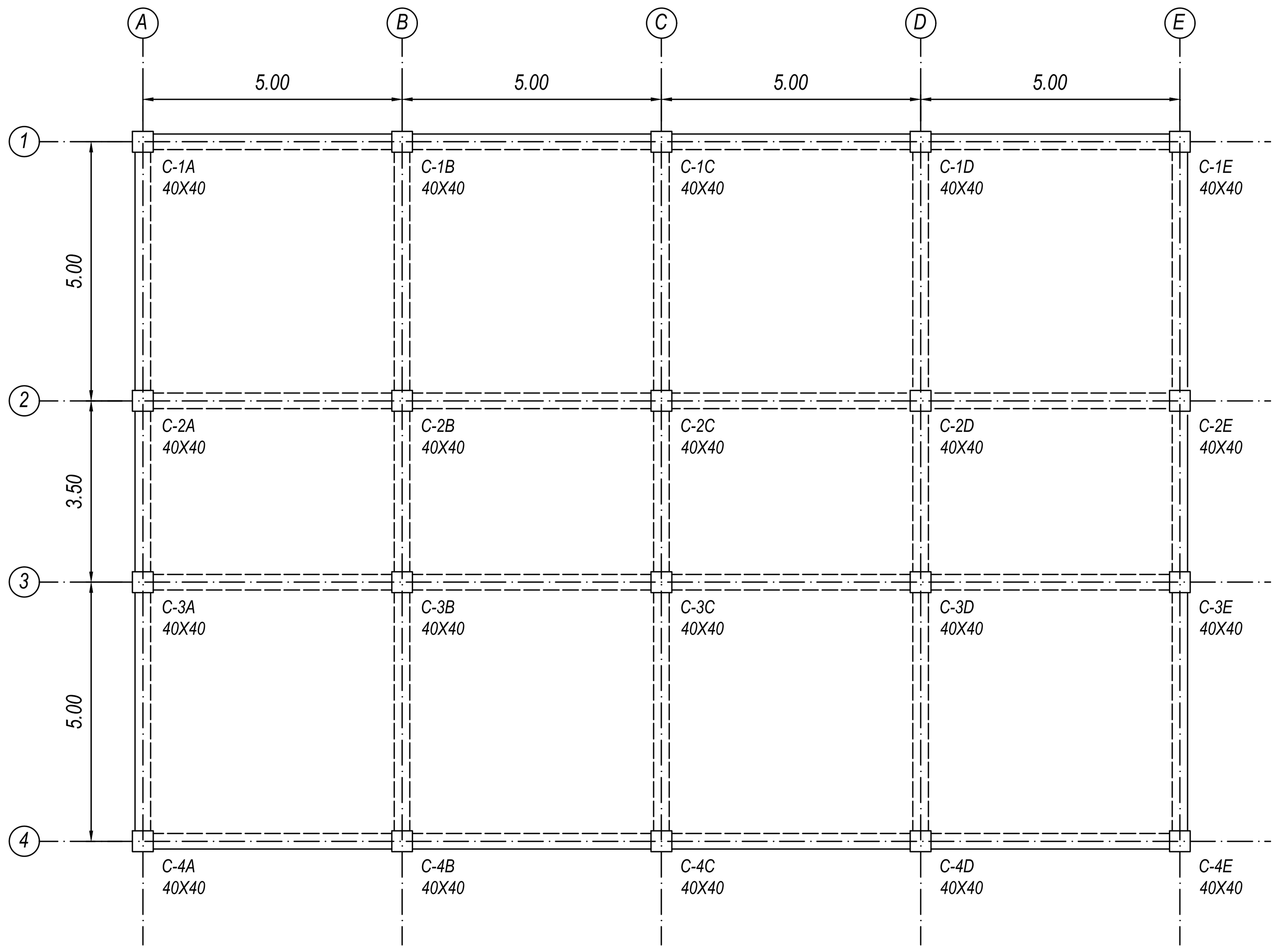
CONVENTIONAL STRUCTURE DESIGNED FOR CHILEAN SEISMIC SPECTRUM	
Structural system:	*Three-dimensional reinforced concrete moment-resisting frame*
Design codes:	*NCh433.Of1996-Mod. 2009, NCh430.Of2008*
Material properties:	
Concrete:	*G25*
Reinforcing steel:	*A630-420H*
Material quantities:	
Designed by:	*Sergio Tito Cardozo Nava*
Sheet No.:	*3 of 3* — **Date:** *March 2025*

Structural element	Concrete (m^3)	Reinforcing steel (kg)	Reinforcement ratio (kg/m^3)
Columns	90.75	23980.00	264.24
Beams	176.11	25030.00	142.13
Slabs	132.61	12897.00	97.26
Total	**399.47**	**61907.00**	

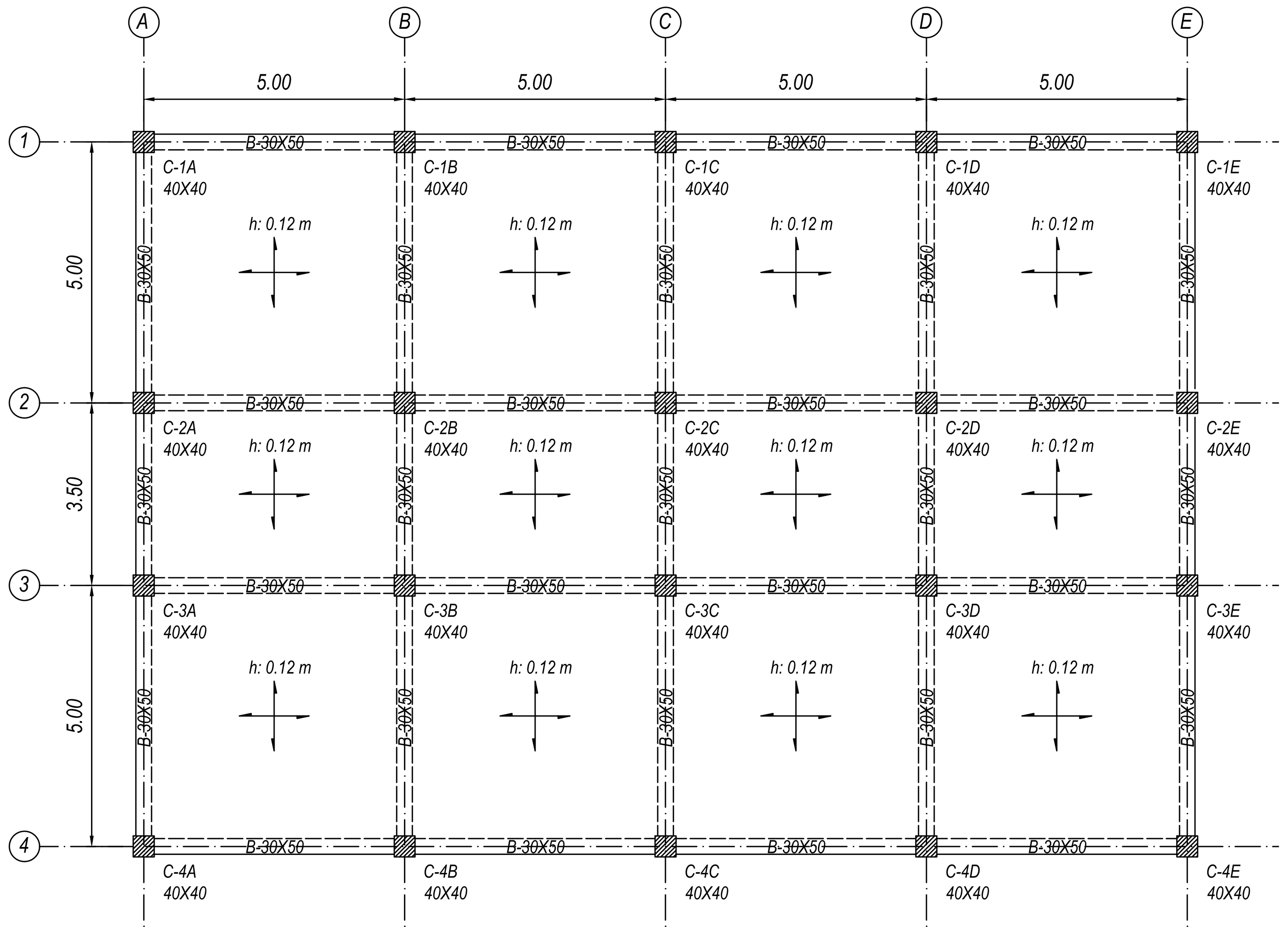
ANNEX B:

PLANS OF CONVENTIONAL STRUCTURE WITH TURKISH SPECTRUM

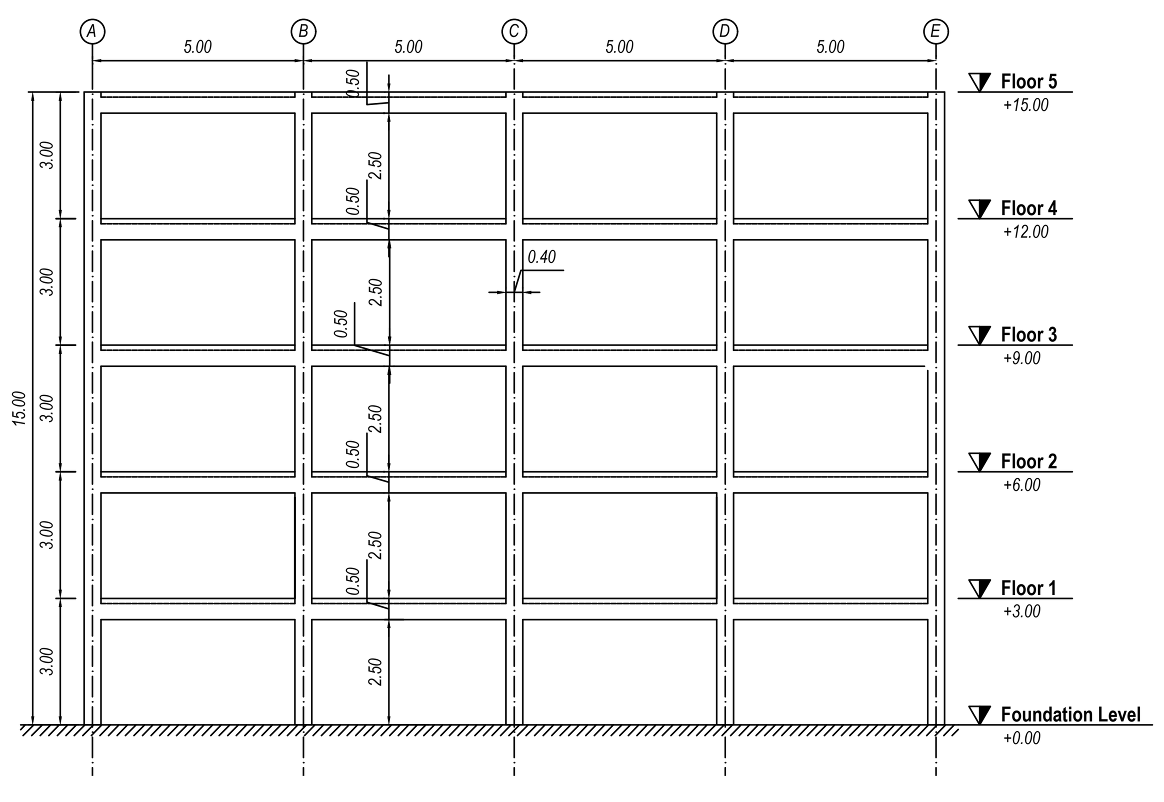
1 Column Staking
Scale 1:100



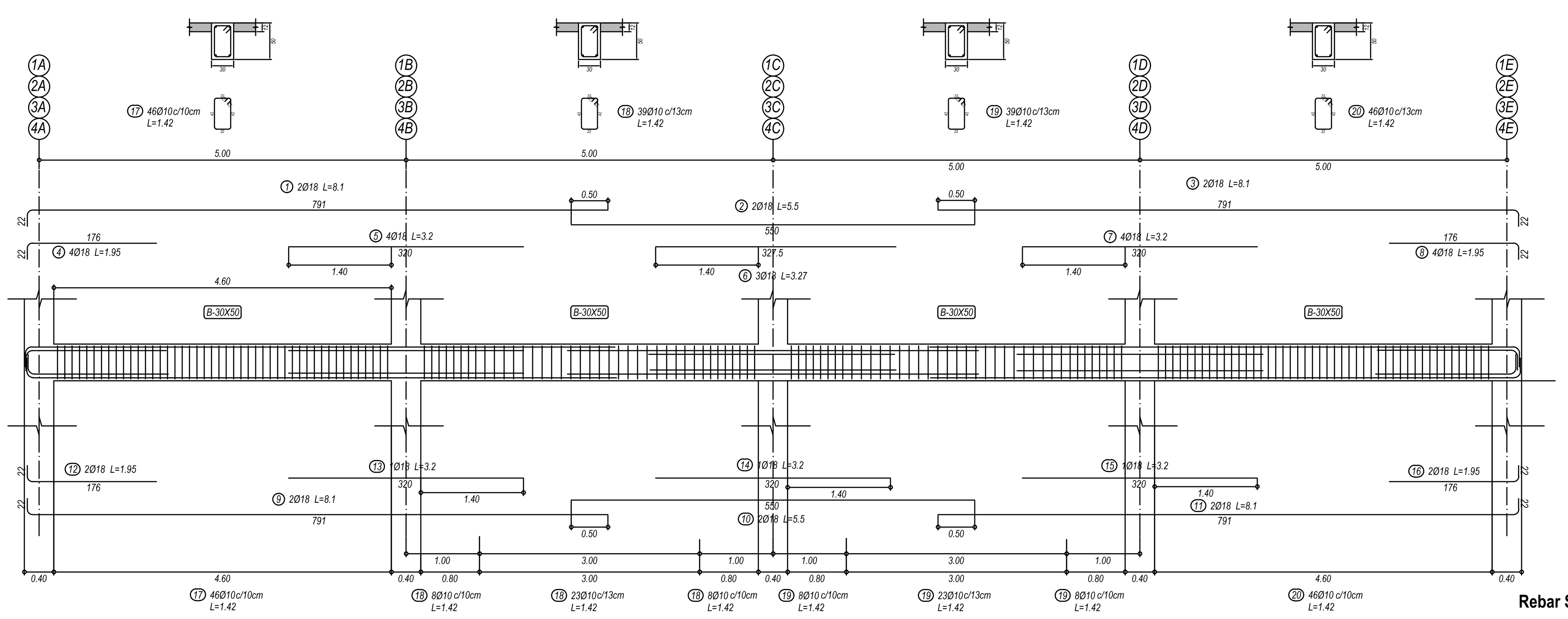
2 Beams (Floor 1, Floor 2 y Floor 3)
Scale 1:100



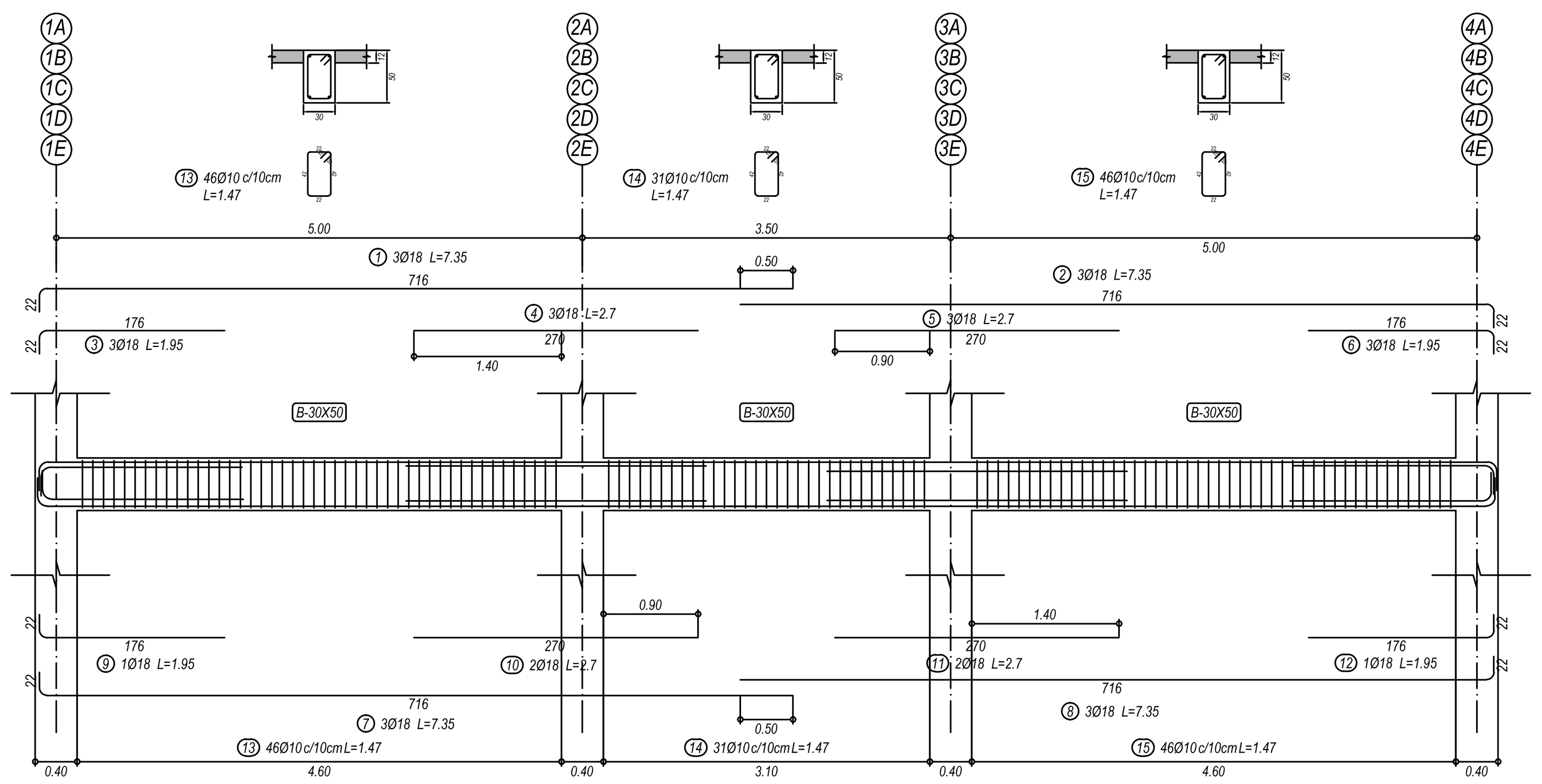
5 Building Elevation
Scale 1:150



3 Beam Rebar Detailing Axes 1,2,3,4 (Floor 1, Floor 2 y Floor 3)
Scale 1:60



4 Beam Rebar Detailing Axes A,B,C,D,E (Floor 1, Floor 2 y Floor 3)
Scale 1:60

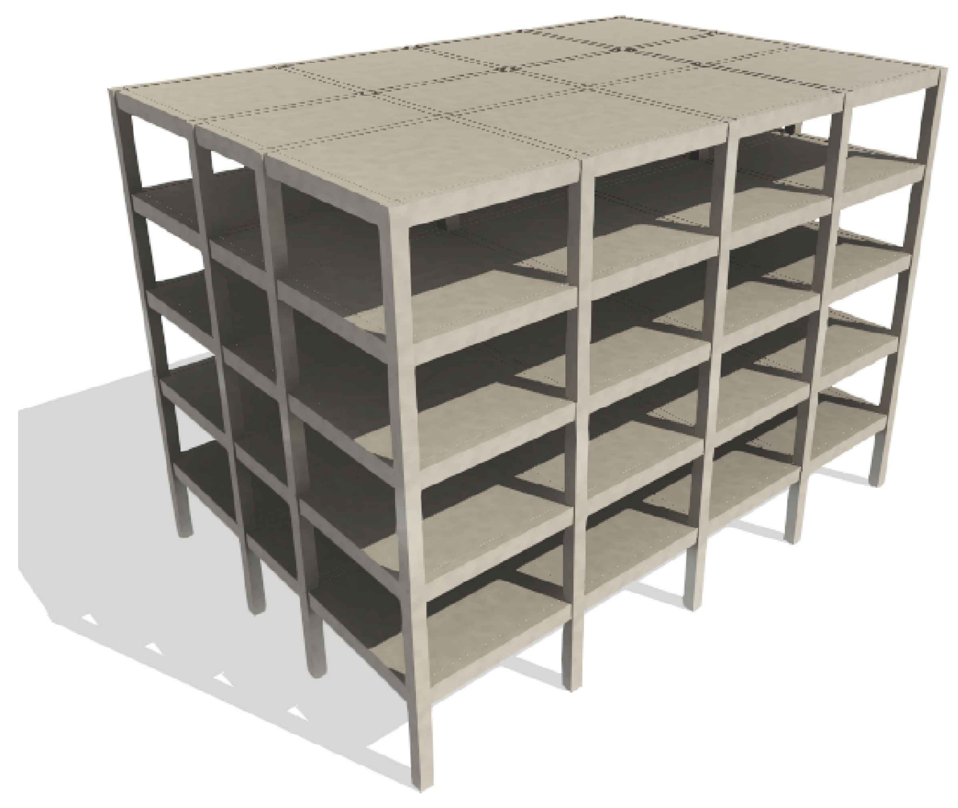


Rebar Schedule de Beams Axis A,B,C,D,E (Floor 1, Floor 2 y Floor 3)

Mark	Ø [mm]	Scheme [cm]	Length [m]	N°	Weight [kg]
1	Ø18	22, 716	7.35	3	44.04
2	Ø18	22, 716	7.35	3	44.04
3	Ø18	22, 176	1.95	3	11.68
4	Ø18	270	2.7	3	16.18
5	Ø18	270	2.7	3	16.18
6	Ø18	22, 176	1.95	3	11.68
7	Ø18	716, 22	7.35	3	44.04

Mark	Ø [mm]	Scheme [cm]	Length [m]	N°	Weight [kg]
8	Ø18	716, 22	7.35	3	44.04
9	Ø18	176, 22	1.95	1	3.89
10	Ø18	270	2.7	2	10.79
11	Ø18	270	2.7	2	10.79
12	Ø18	176, 22	1.95	1	3.89
13	Ø10	42, 10, 10, 22, 22, 42	1.47	46	41.57
14	Ø10	42, 10, 10, 22, 22, 42	1.47	31	28.01
15	Ø10	42, 10, 10, 22, 22, 42	1.47	46	41.57
Total Weight = 372 kg					

6 Structural Scheme
Not to scale



Rebar Schedule de Beams Axis 1,2,3,4 (Floor 1, Floor 2 y Floor 3)

Mark	Ø [mm]	Scheme [cm]	Length [m]	N°	Weight [kg]
1	Ø18	22, 791	8.1	2	32.36
2	Ø18	550	5.5	2	21.97
3	Ø18	22, 791	8.1	2	32.36
4	Ø18	22, 176	1.95	4	15.57
5	Ø18	320	3.2	4	25.57
6	Ø18	327.5	3.27	3	19.63
7	Ø18	320	3.2	4	25.57
8	Ø18	22, 176	1.95	4	15.57
9	Ø18	22, 791	8.1	2	32.36
10	Ø18	550	5.5	2	21.97

Mark	Ø [mm]	Scheme [cm]	Length [m]	N°	Weight [kg]
11	Ø18	22, 791	8.1	2	32.36
12	Ø18	176, 22	1.95	2	7.79
13	Ø18	320	3.2	1	6.39
14	Ø18	320	3.2	1	6.39
15	Ø18	320	3.2	1	6.39
16	Ø18	22, 176	1.95	2	7.79
17	Ø10	42, 7.5, 7.5, 22, 22, 42	1.42	46	40.15
18	Ø10	42, 7.5, 7.5, 22, 22, 42	1.42	39	34.04
19	Ø10	42, 7.5, 7.5, 22, 22, 42	1.42	39	34.04
20	Ø10	42, 7.5, 7.5, 22, 22, 42	1.42	46	40.15
Total Weight = 458 kg					

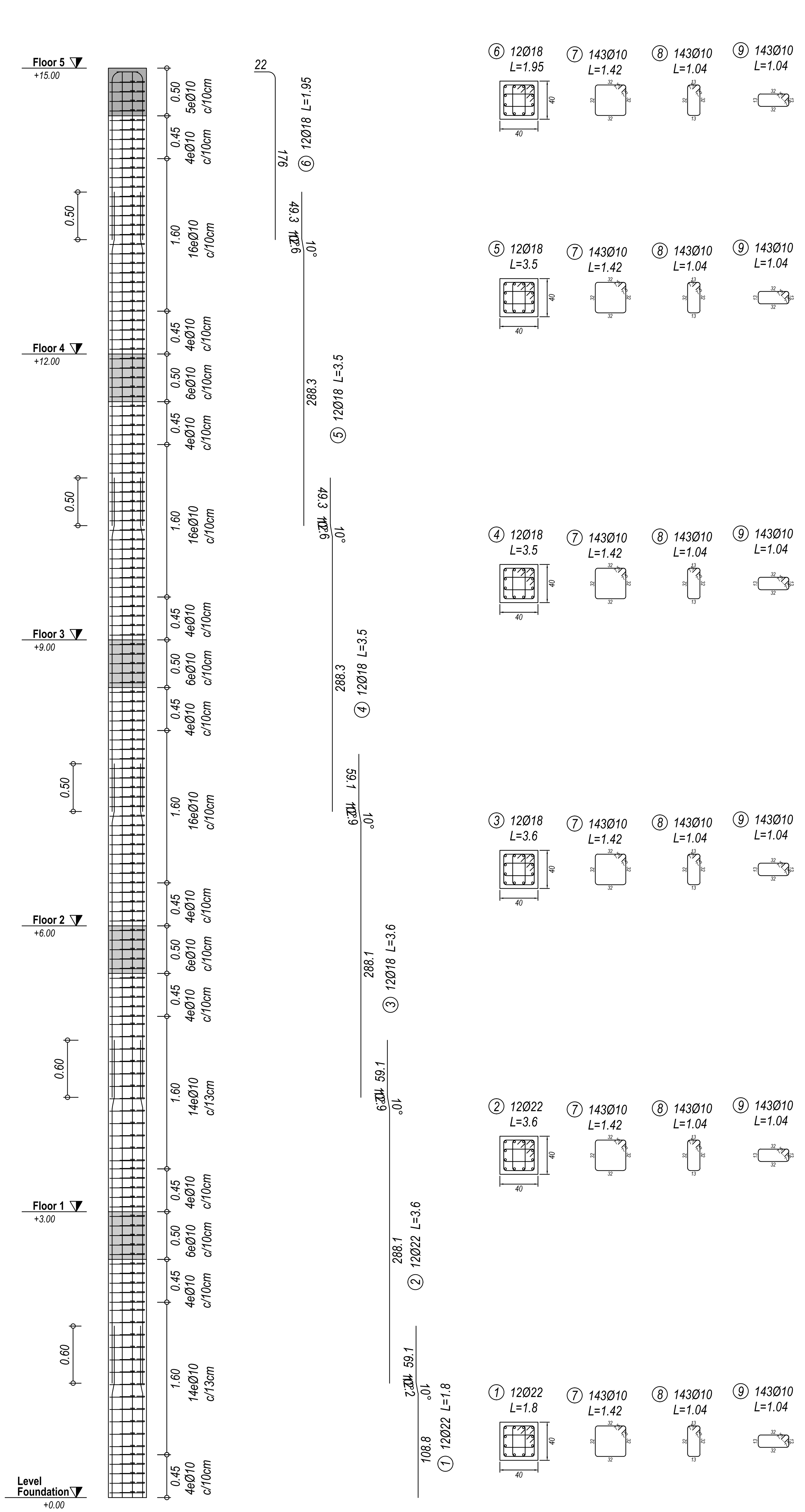
CONVENTIONAL STRUCTURE DESIGNED FOR TURKISH SEISMIC SPECTRUM

Structural system:	*Reinforced concrete moment-resisting frame*
Design codes:	*NCh433.Of1996-Mod. 2009, NCh430.Of2008*
Material properties:	
Concrete:	*G25*
Reinforcing steel:	*A630-420H*
Material quantities:	

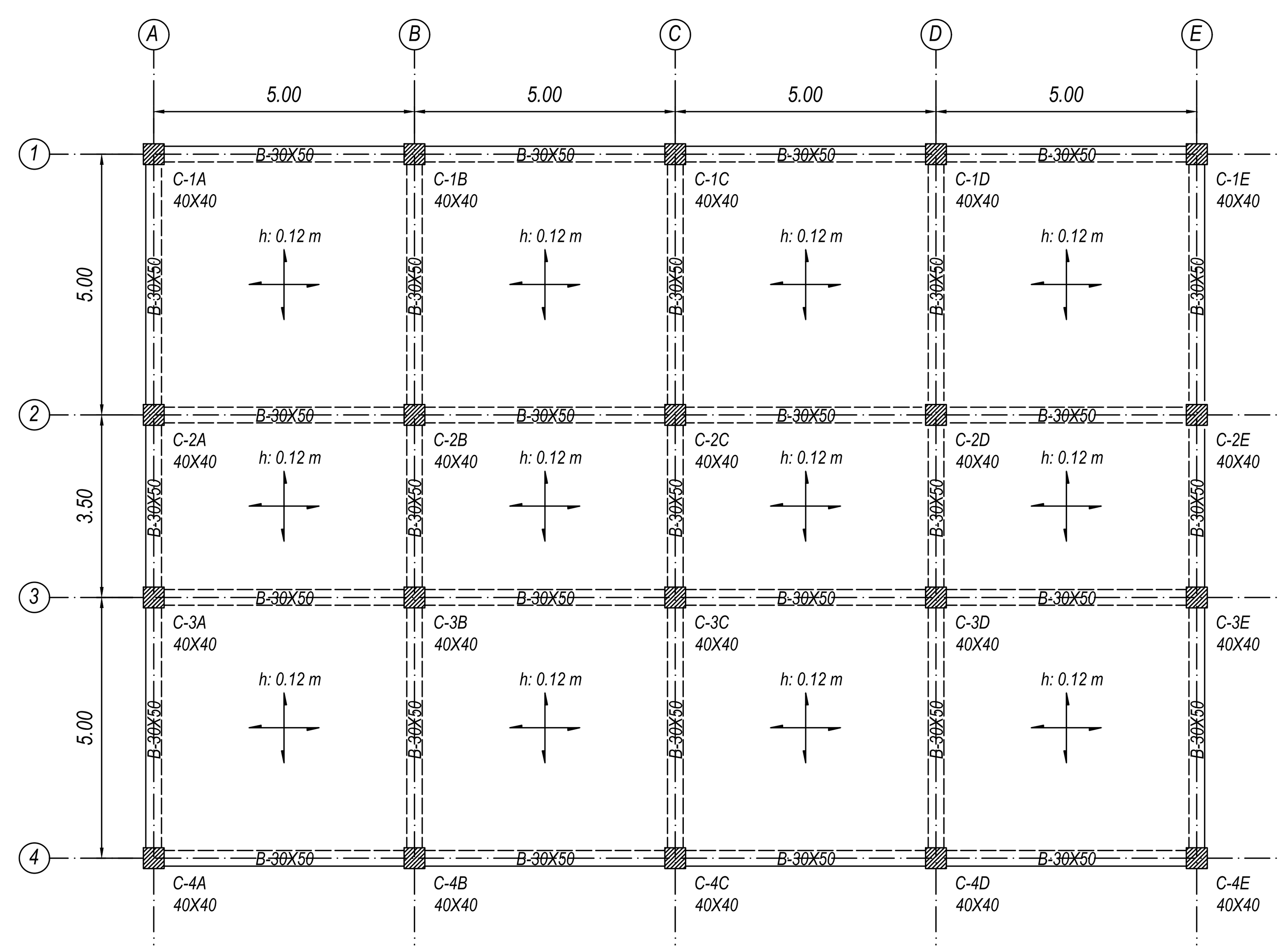
Structural element	Concrete (m^3)	Reinforcing steel (kg)	Reinforcement ratio (kg/m^3)
Columns	48.00	16040.00	334.17
Beams	101.33	15966.00	157.56
Slabs	142.06	12410.00	87.36
Total	**291.39**	**44416.00**	

Designed by:	*Sergio Tito Cardozo Nava*		
Sheet No.:	*1 of 3*	**Date:**	*March 2025*

2 Column Rebar Detailing
Scale 1:30



1 Beams (Floor 4 y Floor 5)
Scale 1:100



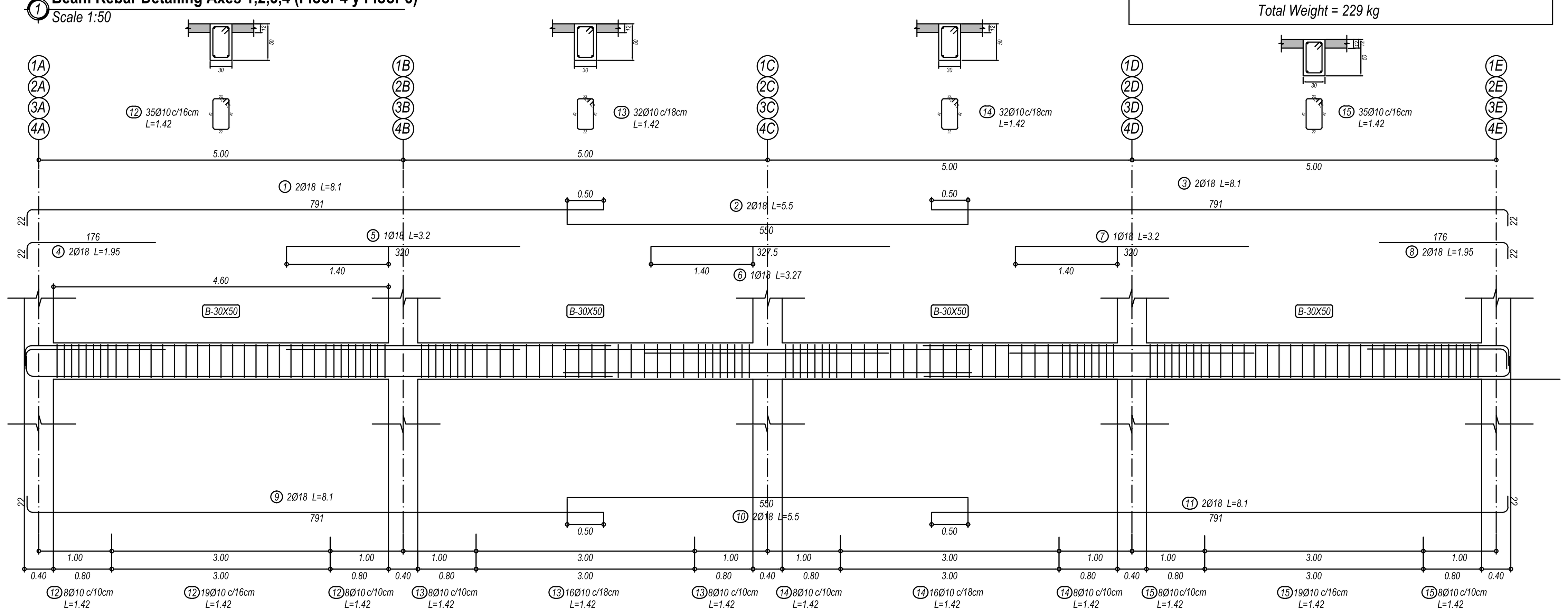
Rebar Schedule de Beams Axis A,B,C,D,E (Floor 4 y Floor 5)

Mark	Ø [mm]	Scheme [cm]	Length [m]	N°	Weight [kg]
1	Ø18	22 / 716	7.35	2	29.36
2	Ø18	22 / 716	7.35	2	29.36
3	Ø18	22 / 176	1.95	2	7.79
4	Ø18	270	2.7	1	5.39
5	Ø18	270	2.7	1	5.39
6	Ø18	22 / 176	1.95	2	7.79
7	Ø18	716 / 22	7.35	2	29.36
8	Ø18	716 / 22	7.35	2	29.36
9	Ø10	42 / 22 / 10 / 10	1.47	35	31.63
10	Ø10	42 / 22 / 10 / 10	1.47	24	21.69
11	Ø10	42 / 22 / 10 / 10	1.47	35	31.63
Total Weight = 229 kg					

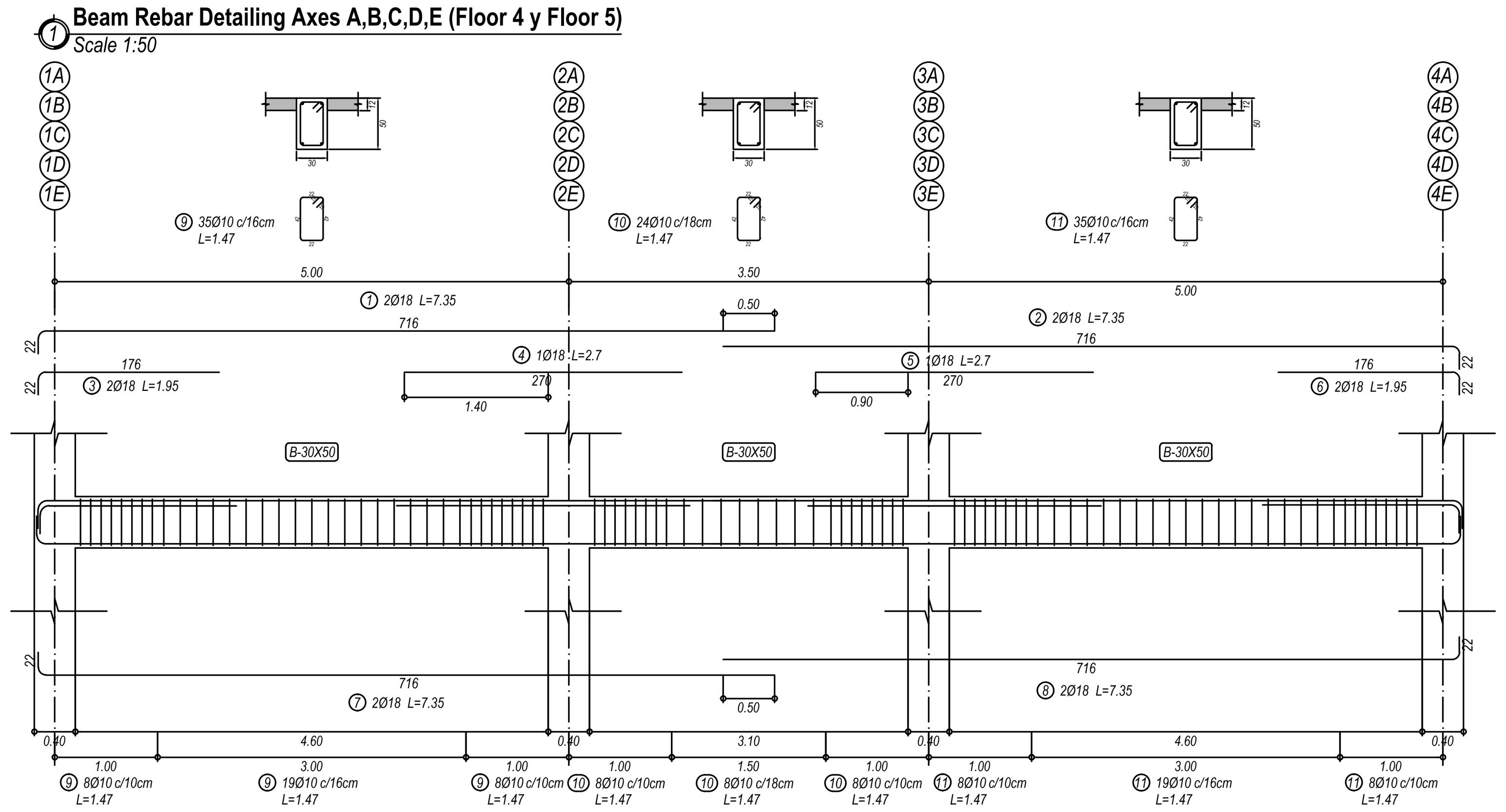
Rebar Schedule de Beams Axis 1,2,3,4 (Floor 4 y Floor 5)

Mark	Ø [mm]	Scheme [cm]	Length [m]	N°	Weight [kg]
1	Ø18	22 / 791	8.1	2	32.36
2	Ø18	550	5.5	2	21.97
3	Ø18	22 / 791	8.1	2	32.36
4	Ø18	22 / 176	1.95	2	7.79
5	Ø18	320	3.2	1	6.39
6	Ø18	327.5	3.27	1	6.54
7	Ø18	320	3.2	1	6.39
8	Ø18	22 / 176	1.95	2	7.79
9	Ø18	22 / 791	8.1	2	32.36
10	Ø18	550	5.5	2	21.97
11	Ø18	22 / 791	8.1	2	32.36
12	Ø10	42 / 22 / 7.5 / 7.5	1.42	35	30.55
13	Ø10	42 / 22 / 7.5 / 7.5	1.42	32	27.93
14	Ø10	42 / 22 / 7.5 / 7.5	1.42	32	27.93
15	Ø10	42 / 22 / 7.5 / 7.5	1.42	35	30.55
Total Weight = 325 kg					

1 Beam Rebar Detailing Axes 1,2,3,4 (Floor 4 y Floor 5)
Scale 1:50



1 Beam Rebar Detailing Axes A,B,C,D,E (Floor 4 y Floor 5)
Scale 1:50



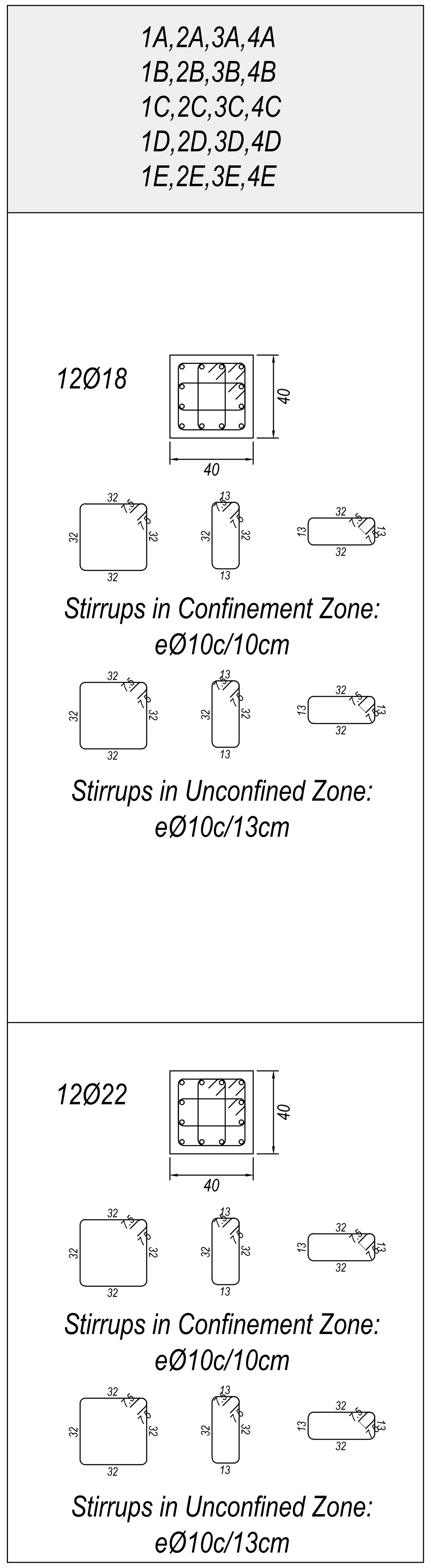
CONVENTIONAL STRUCTURE DESIGNED FOR TURKISH SEISMIC SPECTRUM

Structural system:	Reinforced concrete moment-resisting frame
Design codes:	NCh433.Of1996-Mod. 2009, NCh430.Of2008
Material properties:	
Concrete:	G25
Reinforcing steel:	A630-420H
Material quantities:	

Structural element	Concrete (m³)	Reinforcing steel (kg)	Reinforcement ratio (kg/m³)
Columns	48.00	16040.00	334.17
Beams	101.33	15966.00	157.56
Slabs	142.06	12410.00	87.36
Total	**291.39**	**44416.00**	

Designed by:	Sergio Tito Cardozo Nava		
Sheet No.:	2of 3	**Date:**	March 2025

5 Column Schedule
Scale 1:50



Floor 5

Floor 4

Floor 3

Floor 2

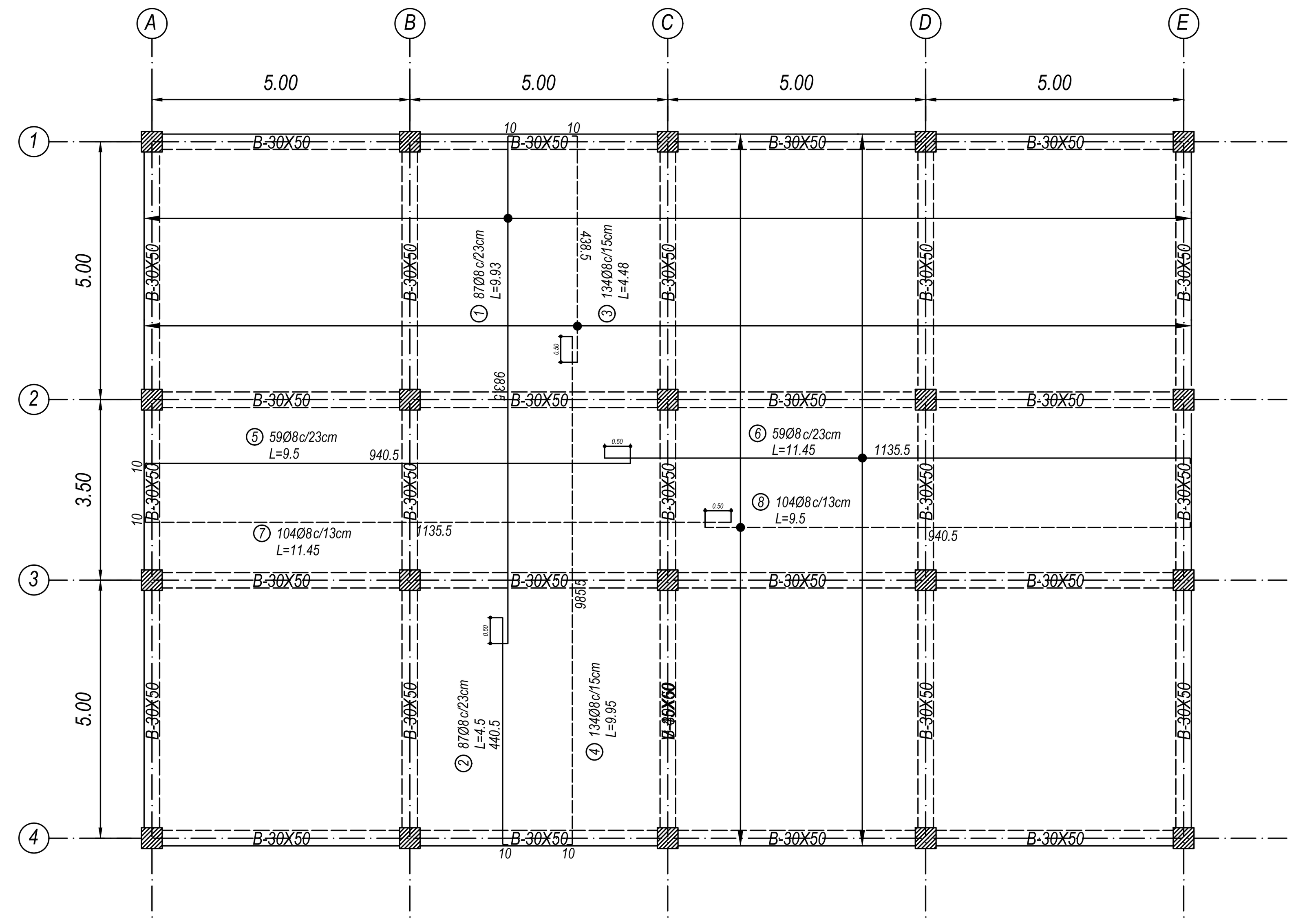
Floor 1

Foundation Level

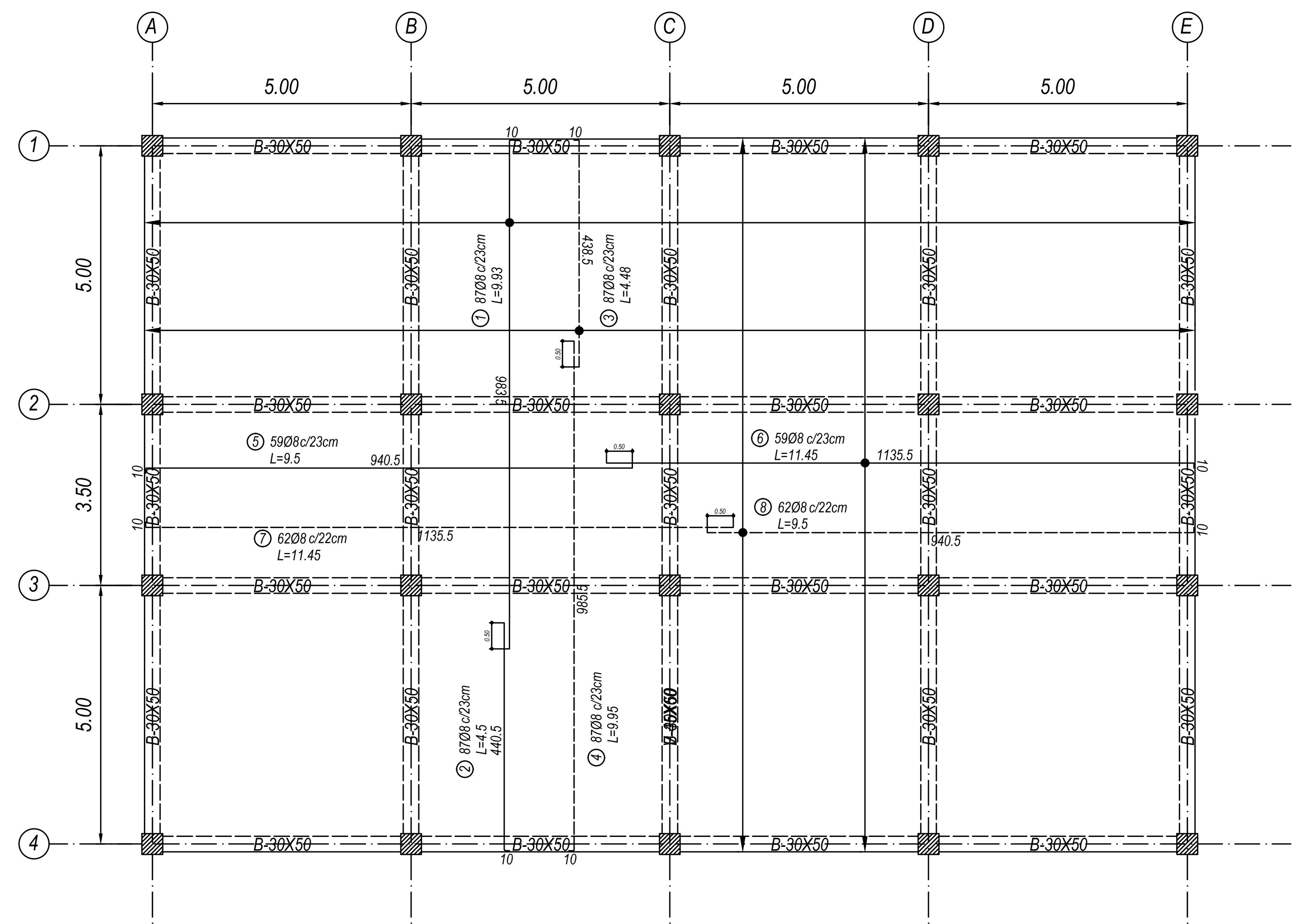
Rebar Schedule Columns

Mark	Ø [mm]	Scheme [cm]	Length [m]	N°	Weight [kg]
1	Ø22	108.8 / 10° / 12.2 / 59.1	1.8	12	64.52
2	Ø22	288.1 / 10° / 12.9 / 59.1	3.6	12	128.98
3	Ø18	288.1 / 10° / 12.9 / 59.1	3.6	12	86.34
4	Ø18	288.3 / 10° / 12.6 / 49.3	3.5	12	83.94
5	Ø18	288.3 / 10° / 12.6 / 49.3	3.5	12	83.94
6	Ø18	176 / 22	1.95	12	46.72
7	Ø10	32 / 32 / 32 / 32 / 7.5 / 7.5	1.42	143	124.81
8	Ø10	32 / 13 / 32 / 13 / 7.5 / 7.5	1.04	143	91.31
9	Ø10	32 / 13 / 32 / 13 / 7.5 / 7.5	1.04	143	91.31
Total Weight = 802 kg					

1 Slab Reinforcement Detail (Floor 1 a Floor 4)
Scale 1:100



2 Slab Reinforcement Detail (Floor 5)
Scale 1:100



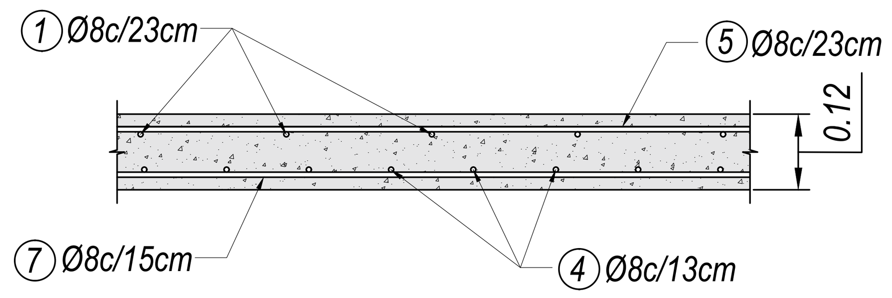
Slab Rebar Schedule (Floor 1 a Floor 4)

Mark	Ø [mm]	Scheme [cm]	Length [m]	N°	Weight [kg]
1	Ø8	10 / 983.5	9.93	87	340.76
2	Ø8	10 / 440.5	4.5	87	154.36
3	Ø8	438.5 / 10	4.48	134	236.69
4	Ø8	10 / 985.5	9.95	134	525.91
5	Ø8	10 / 940.5	9.5	59	221.08
6	Ø8	1135.5 / 10	11.45	59	266.48
7	Ø8	1135.5 / 10	11.45	104	469.72
8	Ø8	940.5 / 10	9.5	104	389.7
Total Weight = 2605 kg					

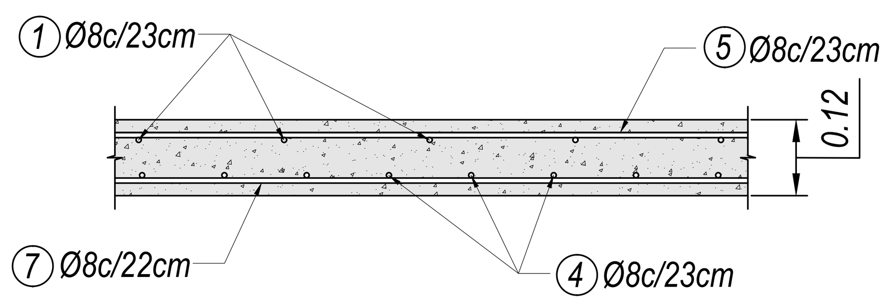
Slab Rebar Schedule (Floor 5)

Mark	Ø [mm]	Scheme [cm]	Length [m]	N°	Weight [kg]
1	Ø8	10 / 983.5	9.93	87	340.76
2	Ø8	10 / 440.5	4.5	87	154.36
3	Ø8	438.5 / 10	4.48	87	153.67
4	Ø8	10 / 985.5	9.95	87	341.45
5	Ø8	10 / 940.5	9.5	59	221.08
6	Ø8	1135.5 / 10	11.45	59	266.48
7	Ø8	1135.5 / 10	11.45	62	280.03
8	Ø8	940.5 / 10	9.5	62	232.32
Total Weight = 1990 kg					

3 (Floor 1 a Floor 4)
Scale 1:10



4 Slab Rebar Detailing (Floor 5)
Scale 1:10



CONVENTIONAL STRUCTURE DESIGNED FOR TURKISH SEISMIC SPECTRUM	
Structural system:	Reinforced concrete moment-resisting frame
Design codes:	NCh433.Of1996-Mod. 2009, NCh430.Of2008
Material properties:	
Concrete:	G25
Reinforcing steel:	A630-420H
Material quantities:	

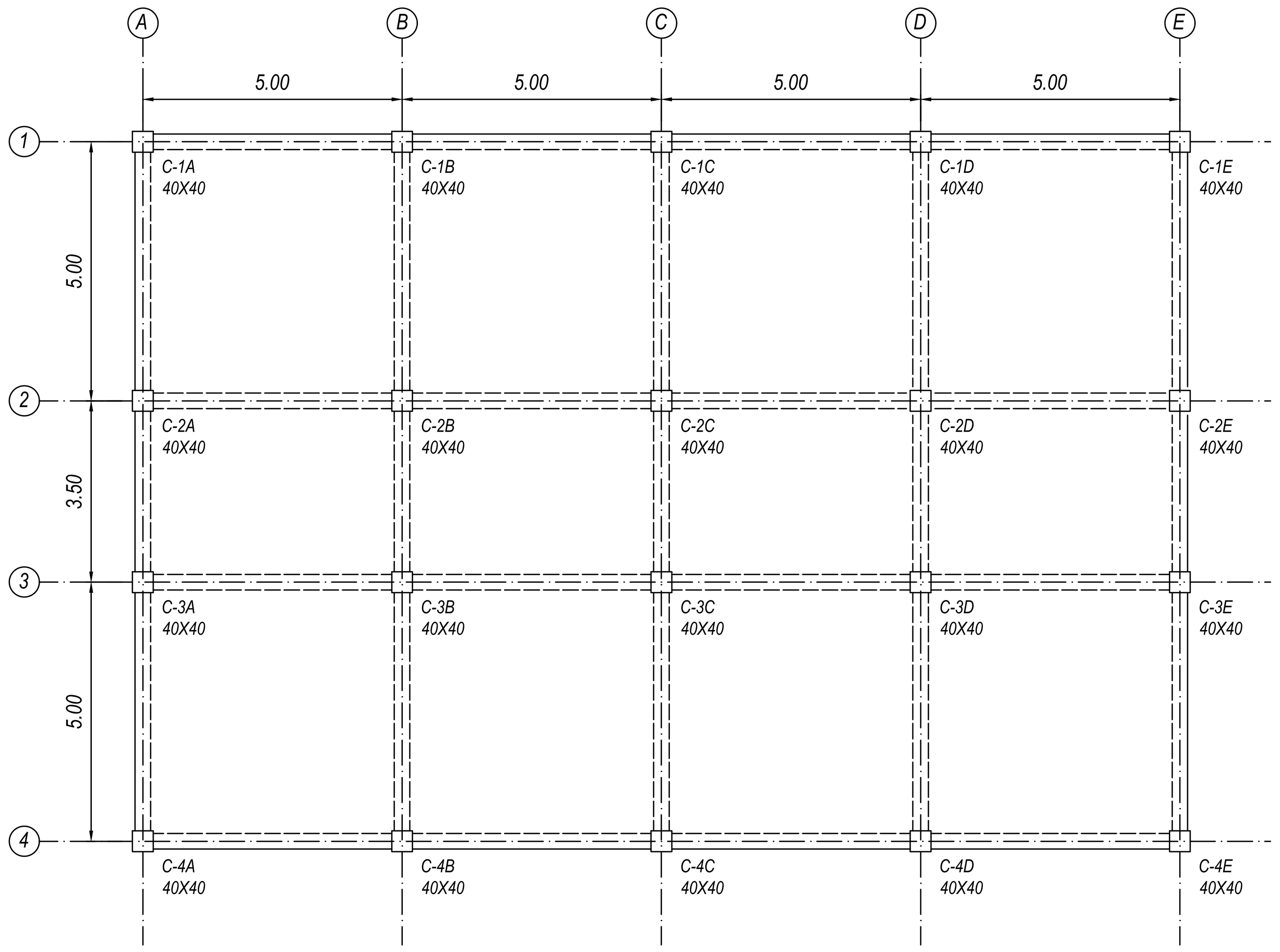
Structural element	Concrete (m^3)	Reinforcing steel (kg)	Reinforcement ratio (kg/m^3)
Columns	48.00	16040.00	334.17
Beams	101.33	15966.00	157.56
Slabs	142.06	12410.00	87.36
Total	**291.39**	**44416.00**	

Designed by:	Sergio Tito Cardozo Nava		
Sheet No.:	3of 3	**Date:**	March 2025

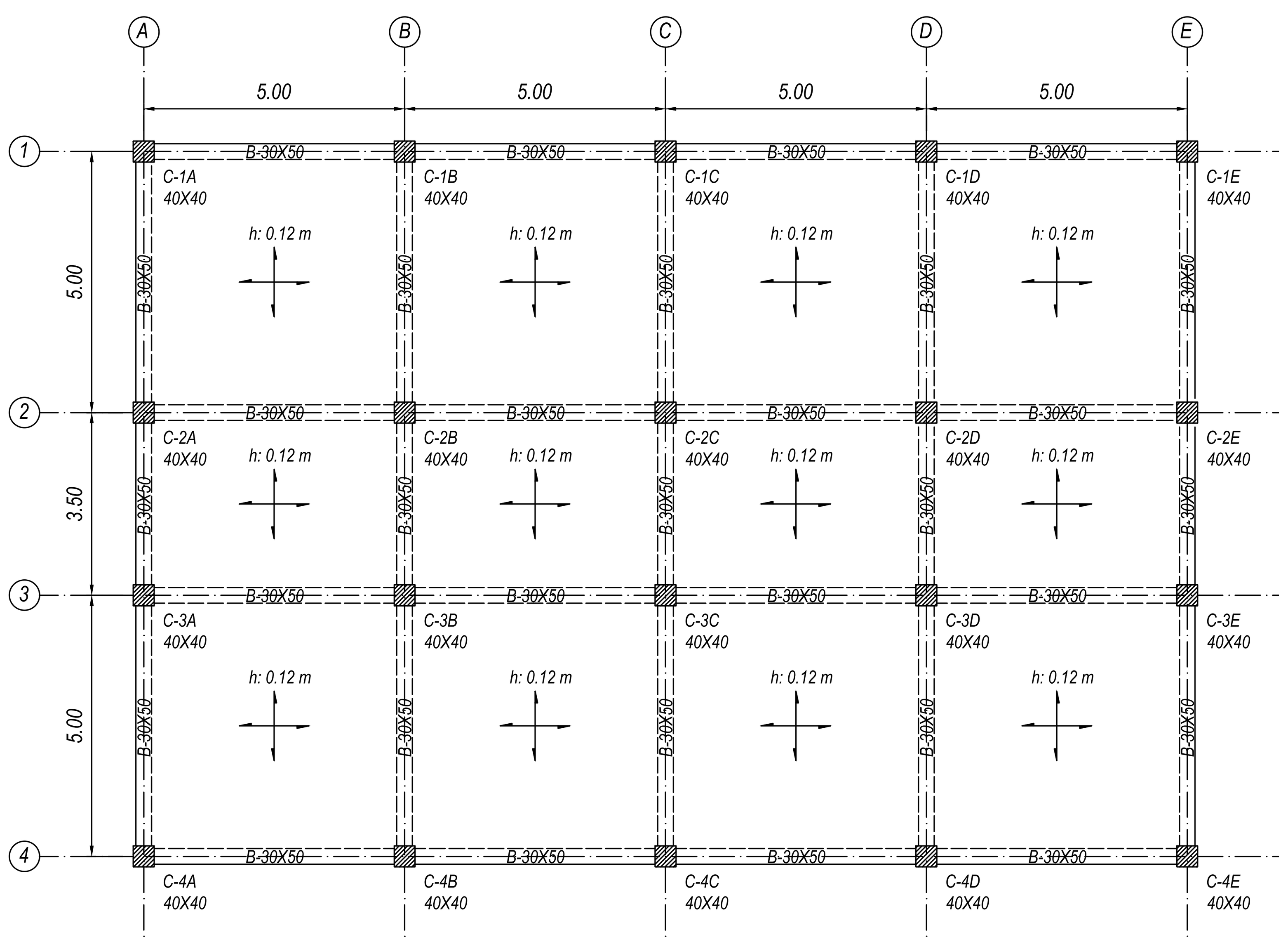
ANNEX C:

PLANS OF ISOLATED STRUCTURE

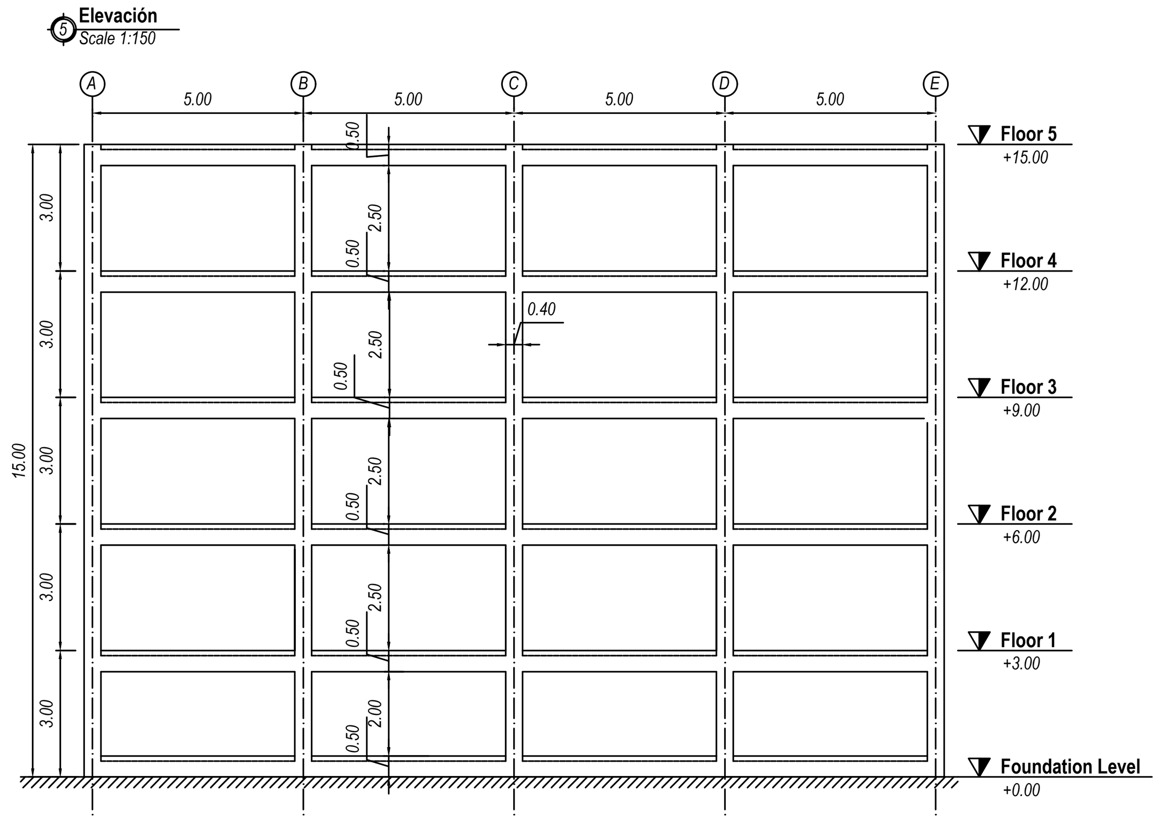
① Column Staking
Scale 1:100



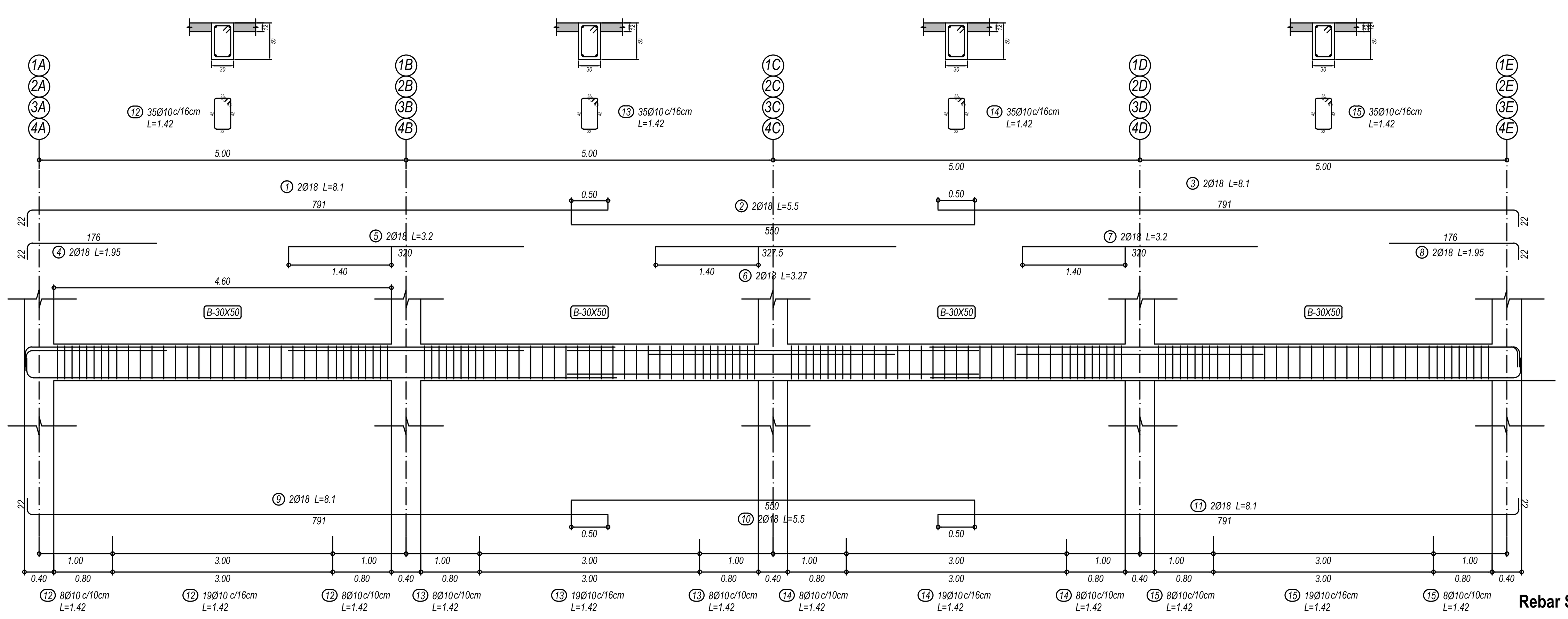
② Vigas (Floor 0,Floor 1 y Floor 2)
Scale 1:100



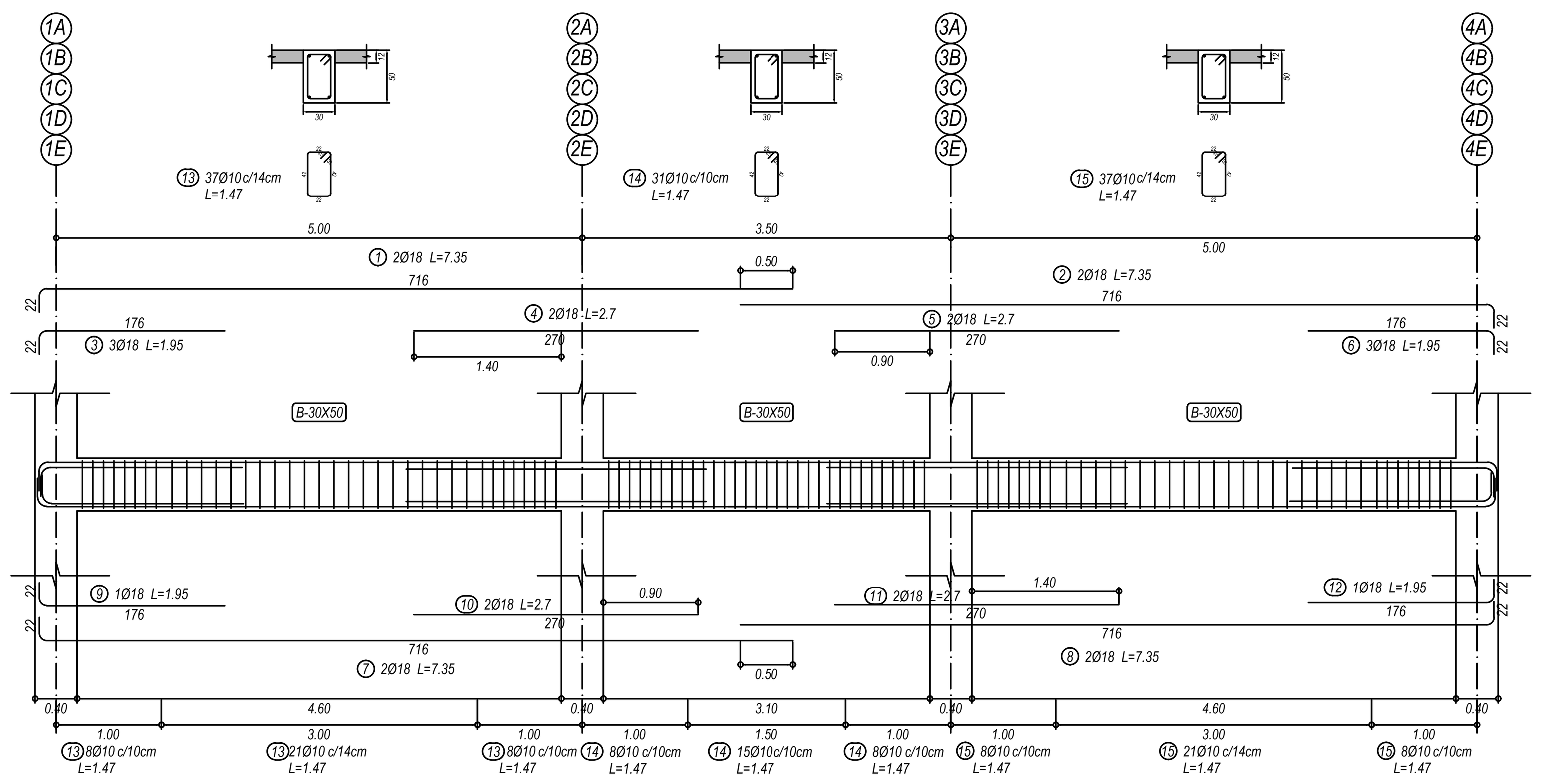
⑤ Elevación
Scale 1:150



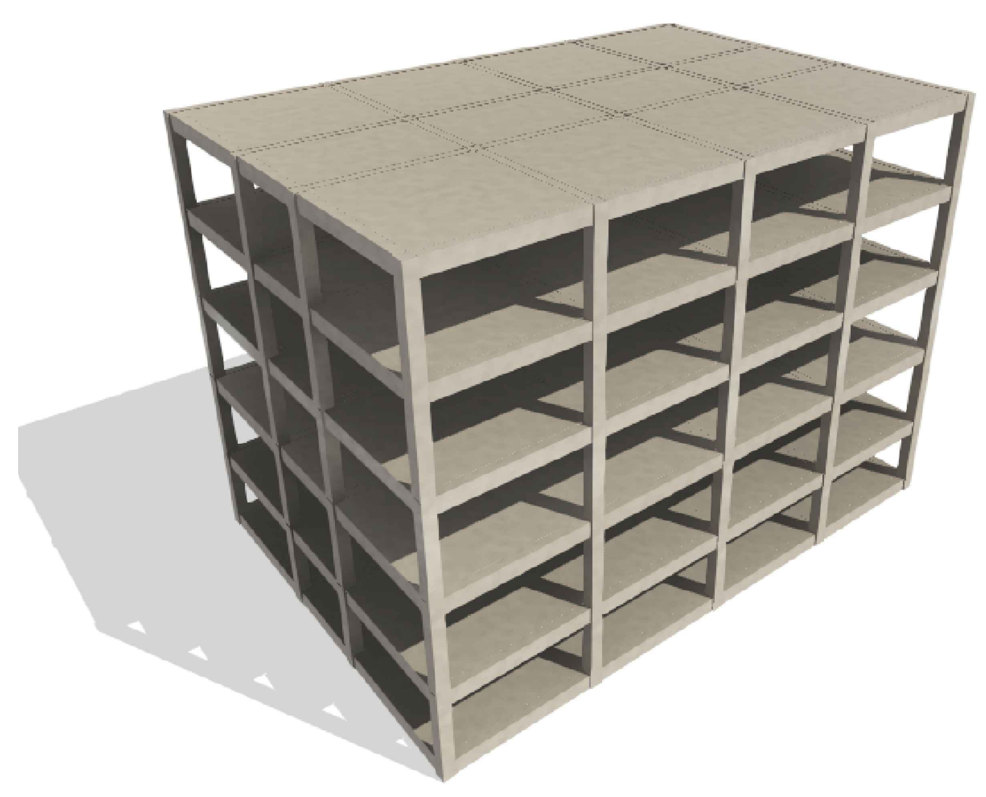
③ Beam Rebar Detailing Axes 1,2,3,4 (Floor 0, Floor 1 y Floor 2)
Scale 1:60



④ Beam Rebar Detailing Axes A,B,C,D,E (Floor 0, Floor 1 y Floor 2)
Scale 1:60



⑥ Esquema de la estrustra
Sin escala



Rebar Schedule de Beams Axis 1,2,3,4 (Floor 0, Floor 1 y Floor 2)

Mark	Ø [mm]	Scheme [cm]	Length [m]	N°	Weigth [kg]
1	Ø18	22 / 791	8.1	2	32.36
2	Ø18	550	5.5	2	21.97
3	Ø18	22 / 791	8.1	2	32.36
4	Ø18	22 / 176	1.95	2	7.79
5	Ø18	320	3.2	2	12.78
6	Ø18	327.5	3.27	2	13.08
7	Ø18	320	3.2	2	12.78
8	Ø18	22 / 176	1.95	2	7.79

Mark	Ø [mm]	Scheme [cm]	Length [m]	N°	Weigth [kg]
9	Ø18	22 / 791	8.1	2	32.36
10	Ø18	550	5.5	2	21.97
11	Ø18	22 / 791	8.1	2	32.36
12	Ø10	42 / 7.5 / 7.5 / 22 / 22 / 42	1.42	35	30.55
13	Ø10	42 / 7.5 / 7.5 / 22 / 22 / 42	1.42	35	30.55
14	Ø10	42 / 7.5 / 7.5 / 22 / 22 / 42	1.42	35	30.55
15	Ø10	42 / 7.5 / 7.5 / 22 / 22 / 42	1.42	35	30.55
Total Weigth = 350 kg					

Rebar Schedule de Beams Axis A,B,C,D,E (Floor 0, Floor 1 y Floor 2)

Mark	Ø [mm]	Scheme [cm]	Length [m]	N°	Weigth [kg]
1	Ø18	22 / 716	7.35	2	29.36
2	Ø18	22 / 716	7.35	2	29.36
3	Ø18	22 / 176	1.95	3	11.68
4	Ø18	270	2.7	2	10.79
5	Ø18	270	2.7	2	10.79
6	Ø18	22 / 176	1.95	3	11.68
7	Ø18	716 / 22	7.35	2	29.36

Mark	Ø [mm]	Scheme [cm]	Length [m]	N°	Weigth [kg]
8	Ø18	716 / 22	7.35	2	29.36
9	Ø18	176 / 22	1.95	1	3.89
10	Ø18	270	2.7	2	10.79
11	Ø18	270	2.7	2	10.79
12	Ø18	176 / 22	1.95	1	3.89
13	Ø10	42 / 10 / 10 / 22 / 22 / 42	1.47	37	33.43
14	Ø10	42 / 10 / 10 / 22 / 22 / 42	1.47	31	28.01
15	Ø10	42 / 10 / 10 / 22 / 22 / 42	1.47	37	33.43
Total Weigth = 287 kg					

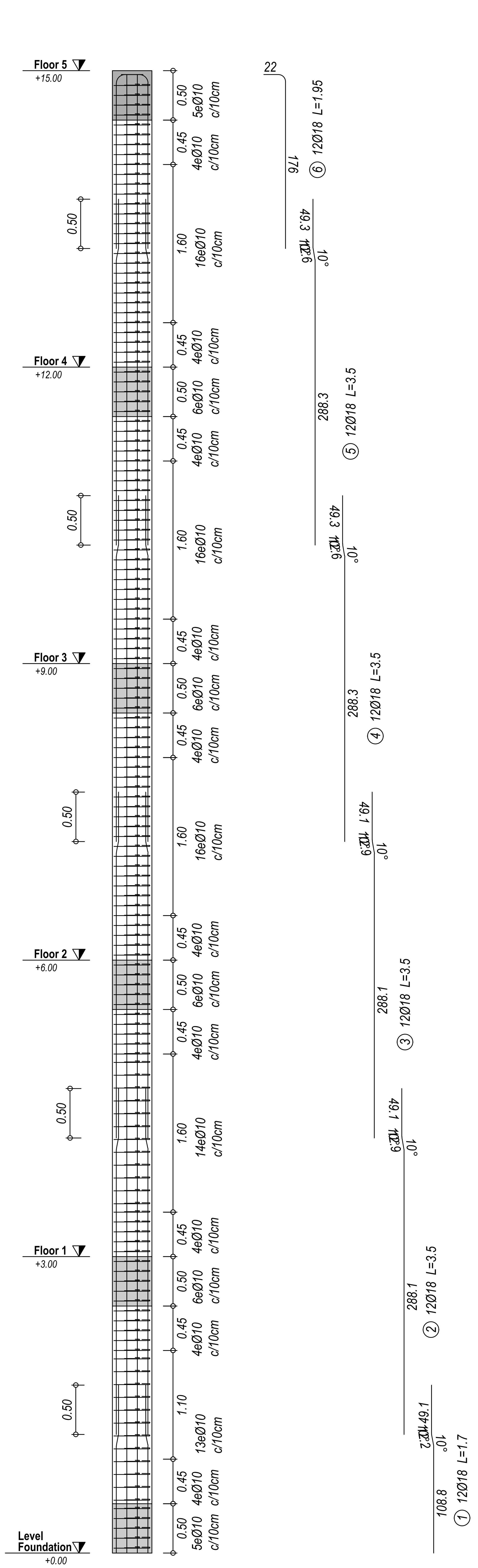
ISOLATED STRUCTURE

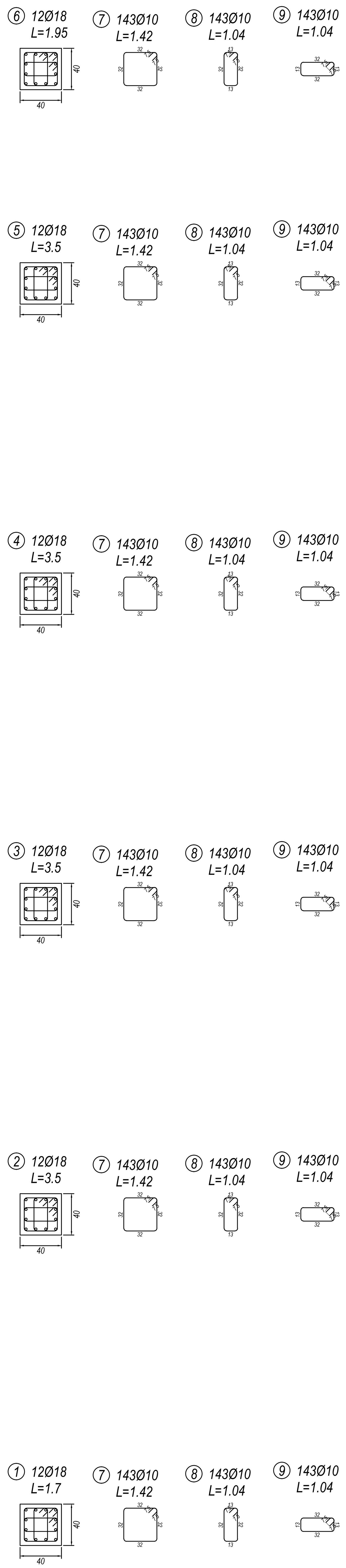
Structural system:	*Reinforced concrete moment-resisting frame with seismic isolation*
Design codes:	*NCh2745:2013, NCh433.Of1996-Mod. 2009, NCh430.Of2008*
Material properties:	
Concrete:	*G25*
Reinforcing steel:	*A630-420H*
Material quantities:	

Structural element	Concrete (m^3)	Reinforcing steel (kg)	Reinforcement ratio (kg/m^3)
Columns	48.00	14620.00	304.58
Beams	121.59	15552.00	127.91
Slabs	170.47	15015.00	88.08
Total	**340.06**	**45187.00**	

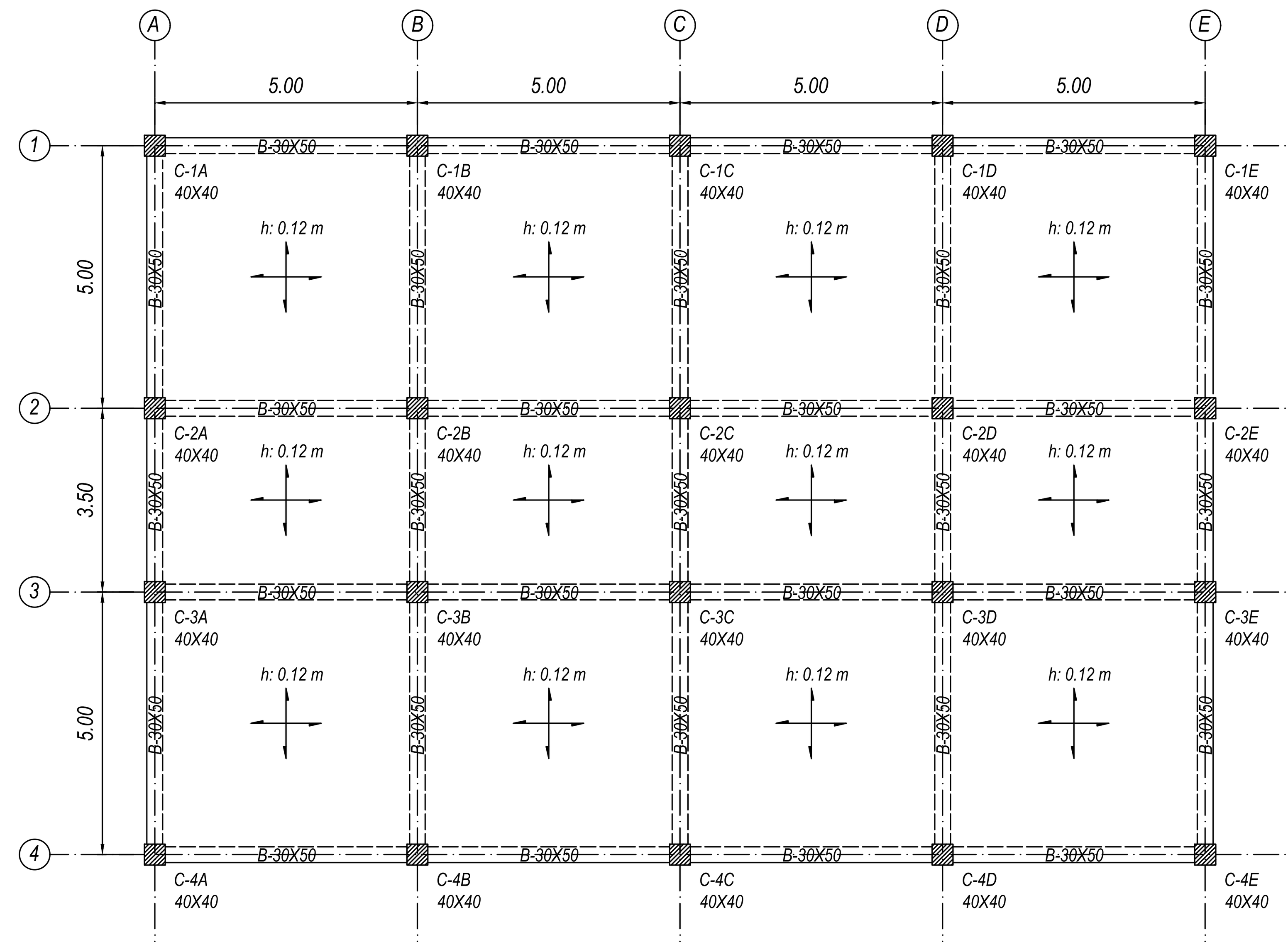
Designed by:	*Sergio Tito Cardozo Nava*		
Sheet No.:	*1 of 3*	**Date:**	*March 2025*

2 Column Rebar Detailing
Scale 1:30





1 Vigas (Floor 3, Floor 4 y Floor 5)
Scale 1:100



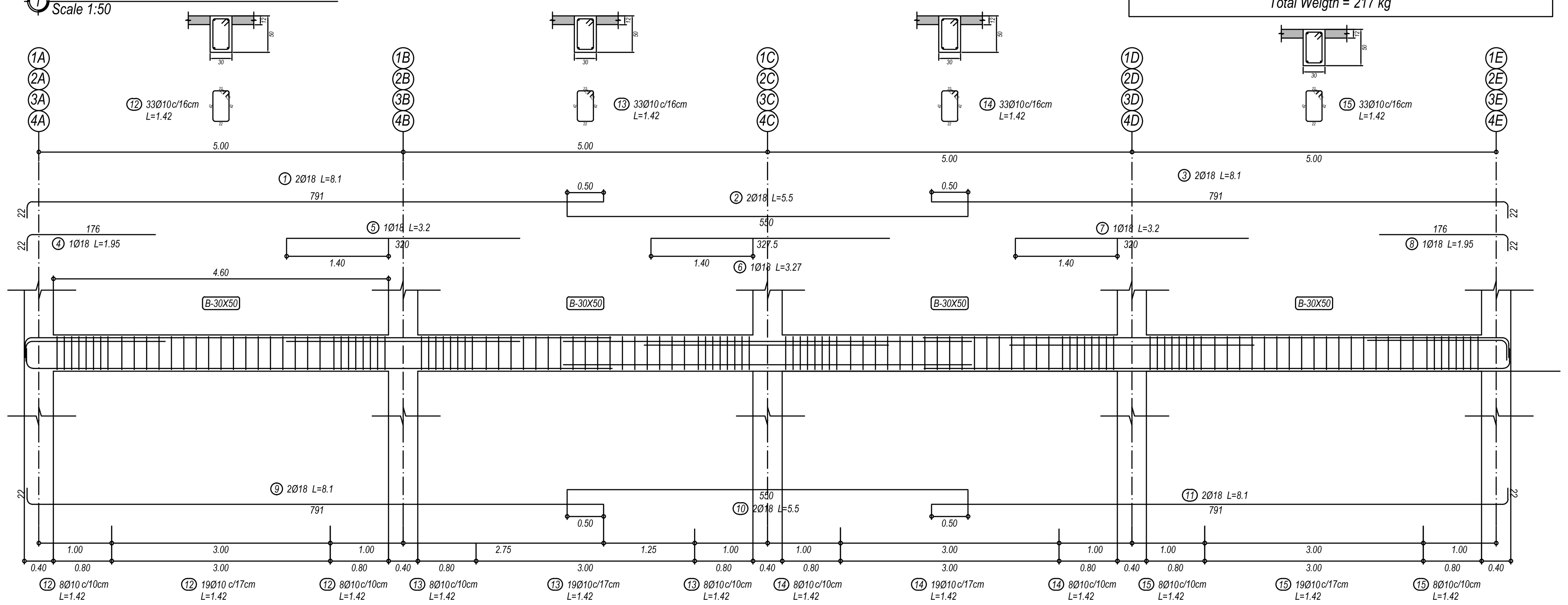
ar Schedule de Beams Axis A,B,C,D,E (Floor3, Floor 4 y Floor 5)

Mark	Ø [mm]	Scheme [cm]	Length [m]	N°	Weigth [kg]
1	Ø18	22, 716	7.35	2	29.36
2	Ø18	22, 716	7.35	2	29.36
3	Ø18	22, 176	1.95	1	3.89
4	Ø18	270	2.7	1	5.39
5	Ø18	270	2.7	1	5.39
6	Ø18	22, 176	1.95	1	3.89
7	Ø18	716, 22	7.35	2	29.36
8	Ø18	716, 22	7.35	2	29.36
9	Ø10	42, 22, 10	1.47	33	29.82
10	Ø10	42, 22, 10	1.47	24	21.69
11	Ø10	42, 22, 10	1.47	33	29.82
Total Weigth = 217 kg					

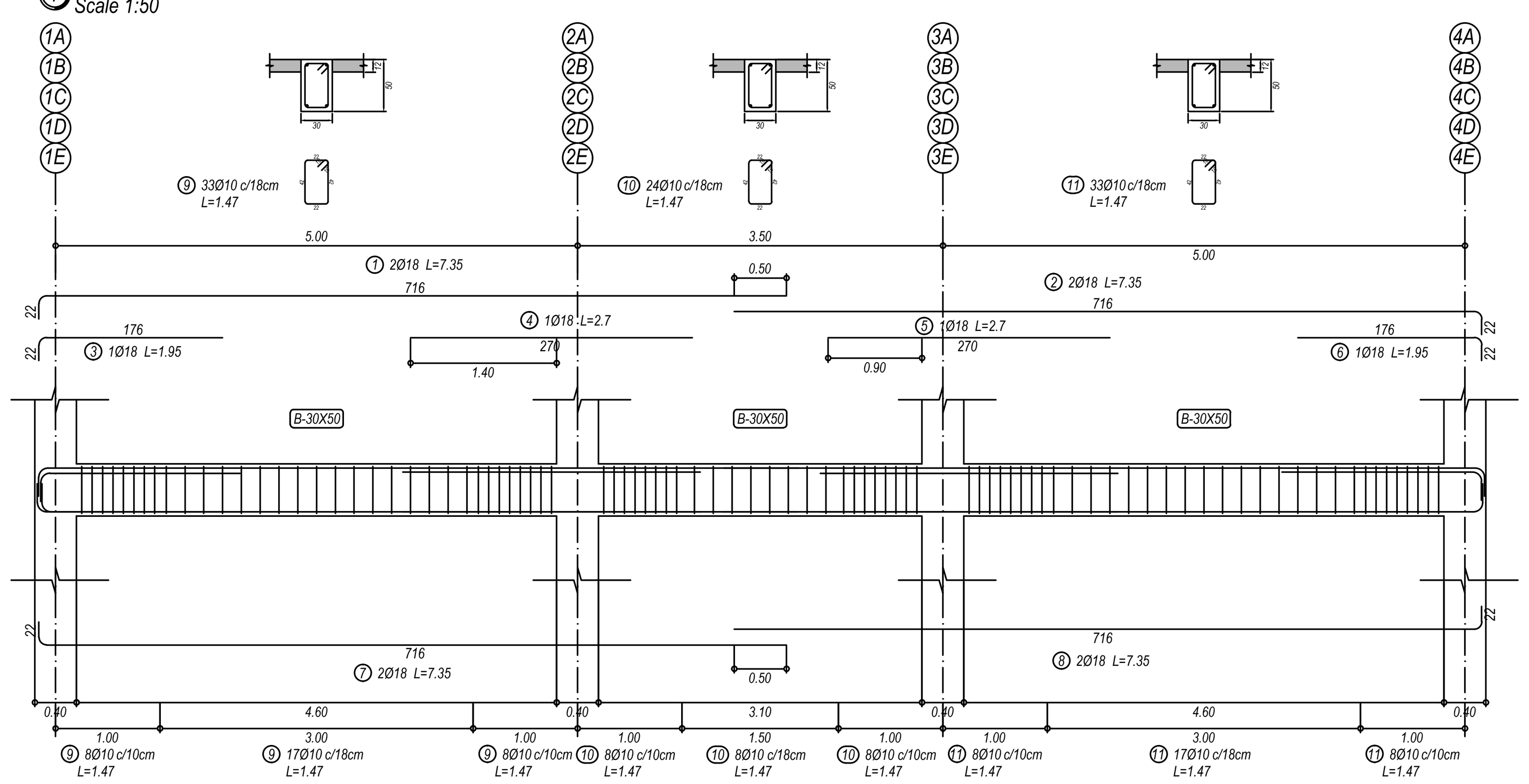
Rebar Schedule de Beams Axis 1,2,3,4 (Floor 3, Floor 4 y Floor 5)

Mark	Ø [mm]	Scheme [cm]	Length [m]	N°	Weigth [kg]
1	Ø18	22, 791	8.1	2	32.36
2	Ø18	550	5.5	2	21.97
3	Ø18	22, 791	8.1	2	32.36
4	Ø18	22, 176	1.95	1	3.89
5	Ø18	320	3.2	1	6.39
6	Ø18	327.5	3.27	1	6.54
7	Ø18	320	3.2	1	6.39
8	Ø18	22, 176	1.95	1	3.89
9	Ø18	22, 791	8.1	2	32.36
10	Ø18	550	5.5	2	21.97
11	Ø18	22, 791	8.1	2	32.36
12	Ø10	42, 22, 7.5	1.42	33	28.8
13	Ø10	42, 22, 7.5	1.42	33	28.8
14	Ø10	42, 22, 7.5	1.42	33	28.8
15	Ø10	42, 22, 7.5	1.42	33	28.8
Total Weigth = 316 kg					

1 Beam Rebar Detailing Axes 1,2,3,4 (Floor 3, Floor 4 y Floor 5)
Scale 1:50



1 Beam Rebar Detailing Axes A,B,C,D,E (Floor 3, Floor 4 y Floor 5)
Scale 1:50



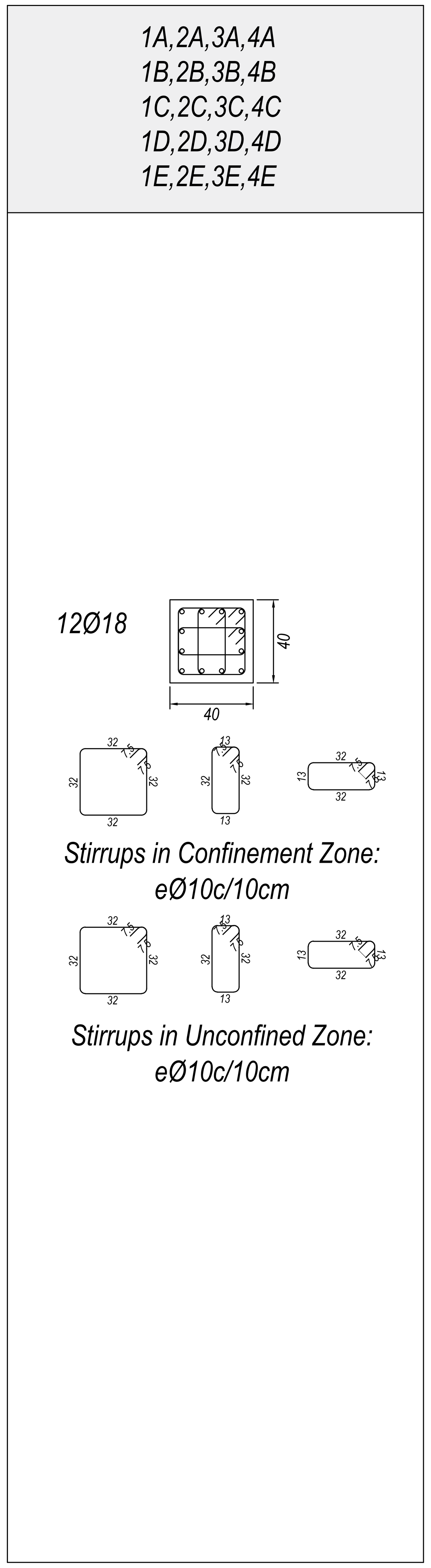
ISOLATED STRUCTURE

Structural system:	*Reinforced concrete moment-resisting frame with seismic isolation*
Design codes:	*NCh2745:2013, NCh433.Of1996-Mod. 2009, NCh430.Of2008*
Material properties:	
Concrete:	*G25*
Reinforcing steel:	*A630-420H*
Material quantities:	

Structural element	Concrete (m³)	Reinforcing steel (kg)	Reinforcement ratio (kg/m³)
Columns	48.00	14620.00	304.58
Beams	121.59	15552.00	127.91
Slabs	170.47	15015.00	88.08
Total	**340.06**	**45187.00**	

Designed by:	*Sergio Tito Cardozo Nava*		
Sheet No.:	*2of 3*	**Date:**	*March 2025*

5 Column Schedule
Scale 1:50



Floor 5

Floor 4

Floor 3

Floor 2

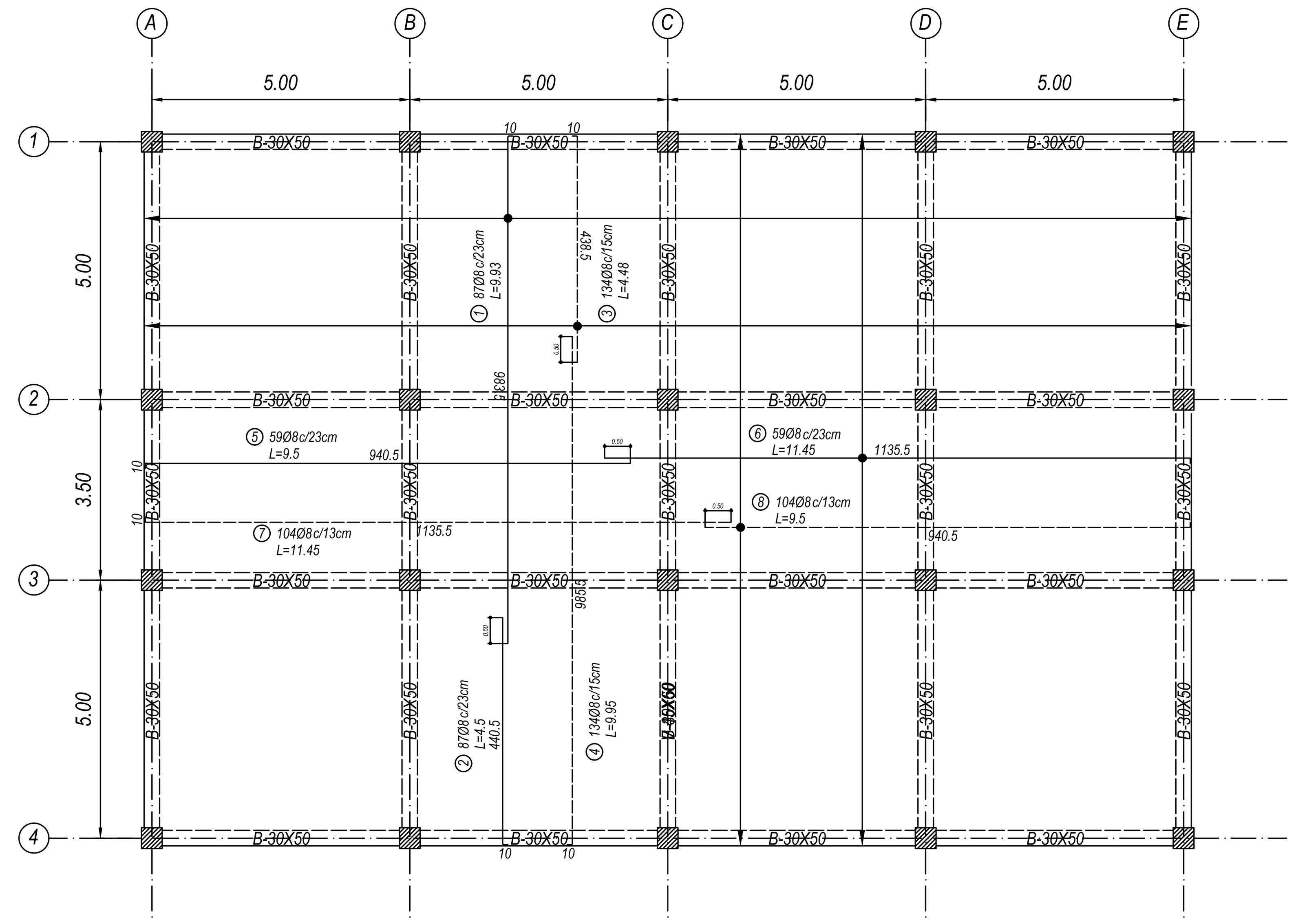
Floor 1

Foundation Level

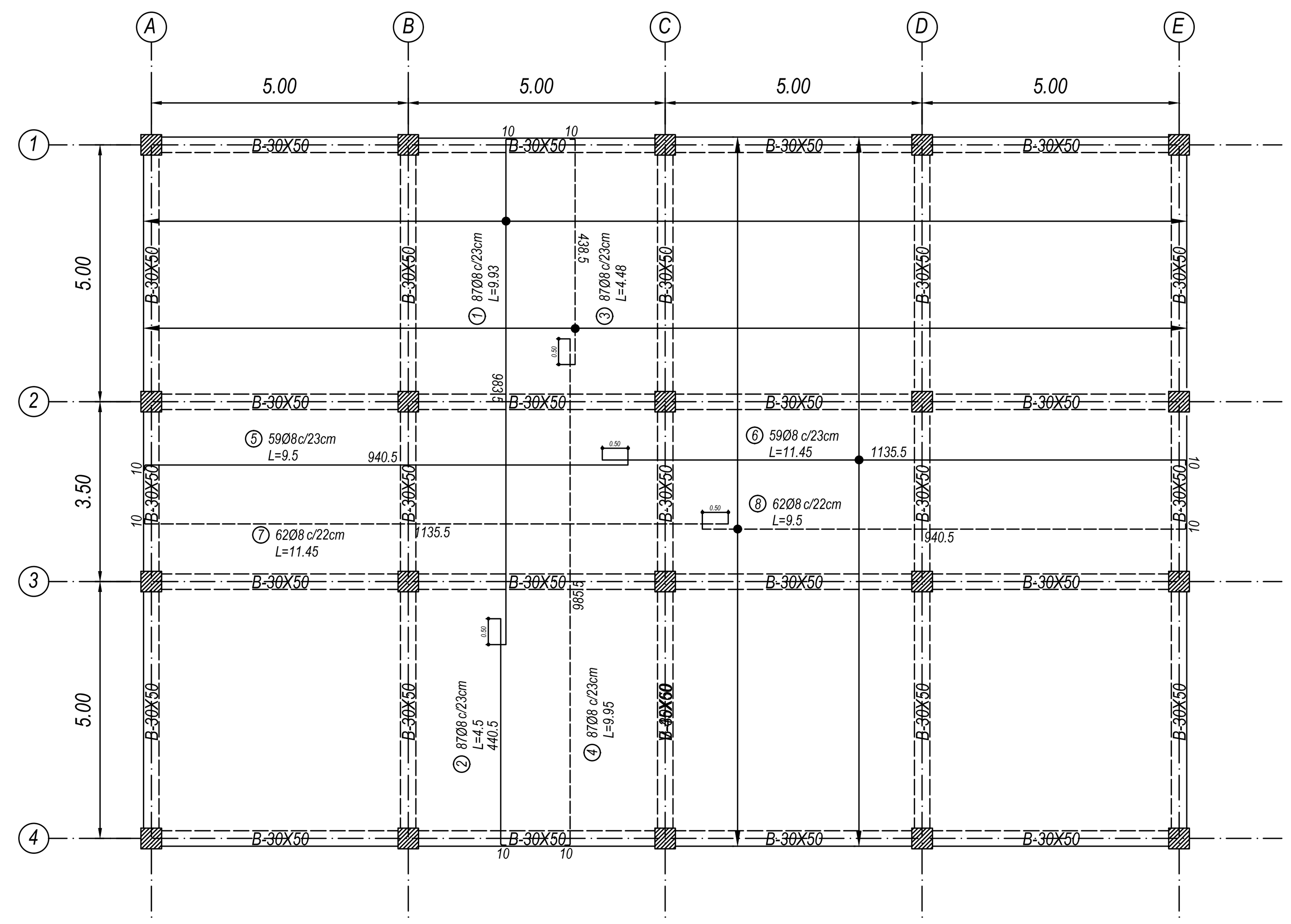
Rebar Schedule Columns

Mark	Ø [mm]	Scheme [cm]	Length [m]	N°	Weigth [kg]
1	Ø18	108.8 – 10° – 12.2 – 49.1	1.7	12	40.8
2	Ø18	288.1 – 10° 12.9 – 49.1	3.5	12	83.94
3	Ø18	288.1 – 10° 12.9 – 49.1	3.5	12	83.94
4	Ø18	288.3 – 10° 12.6 – 49.3	3.5	12	83.94
5	Ø18	288.3 – 10° 12.6 – 49.3	3.5	12	83.94
6	Ø18	176 – 22	1.95	12	46.72
7	Ø10	32 × 32, hooks 7.5	1.42	143	124.81
8	Ø10	32 × 13, hooks 7.5	1.04	143	91.31
9	Ø10	32 × 13, hooks 7.5	1.04	143	91.31
Total Weigth = 731 kg					

1 Slab Reinforcement Detail (Floor 0 a Floor 4)
Scale 1:100



2 Slab Reinforcement Detail (Floor 5)
Scale 1:100



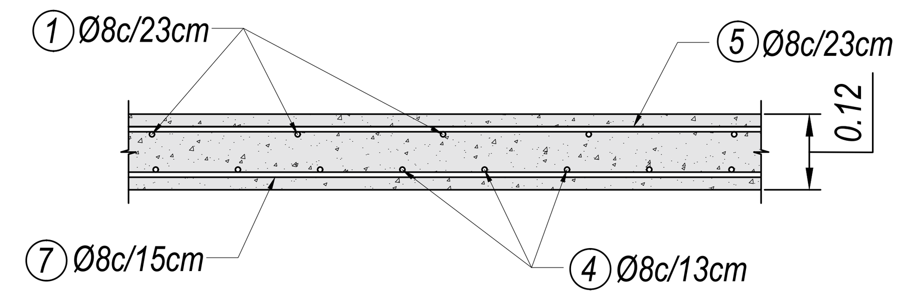
Slab Rebar Schedule (Floor 0 a Floor 4)

Mark	Ø [mm]	Scheme [cm]	Length [m]	N°	Weigth [kg]
1	Ø8	10 – 983.5	9.93	87	340.76
2	Ø8	10 – 440.5	4.5	87	154.36
3	Ø8	438.5 – 10	4.48	134	236.69
4	Ø8	10 – 985.5	9.95	134	525.91
5	Ø8	10 – 940.5	9.5	59	221.08
6	Ø8	1135.5 – 10	11.45	59	266.48
7	Ø8	1135.5 – 10	11.45	104	469.72
8	Ø8	940.5 – 10	9.5	104	389.7
Total Weigth = 2605 kg					

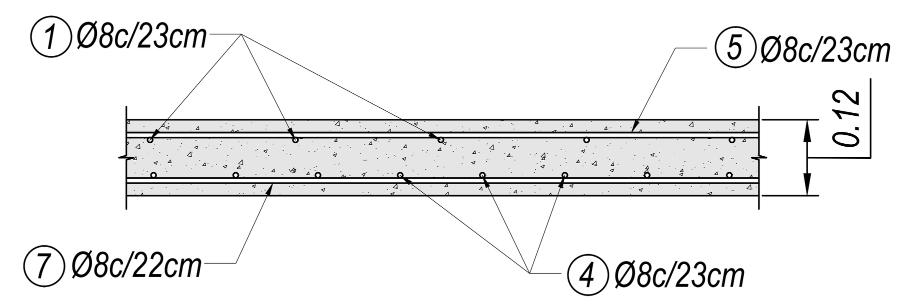
Slab Rebar Schedule (Floor 5)

Mark	Ø [mm]	Scheme [cm]	Length [m]	N°	Weigth [kg]
1	Ø8	10 – 983.5	9.93	87	340.76
2	Ø8	10 – 440.5	4.5	87	154.36
3	Ø8	438.5 – 10	4.48	87	153.67
4	Ø8	10 – 985.5	9.95	87	341.45
5	Ø8	10 – 940.5	9.5	59	221.08
6	Ø8	1135.5 – 10	11.45	59	266.48
7	Ø8	1135.5 – 10	11.45	62	280.03
8	Ø8	940.5 – 10	9.5	62	232.32
Total Weigth = 1990 kg					

3 Slab Rebar Detailing (Floor 0 a Floor 4)
Scale 1:10



4 Slab Rebar Detailing (Floor 5)
Scale 1:10



ISOLATED STRUCTURE	
Structural system:	*Reinforced concrete moment-resisting frame with seismic isolation*
Design codes:	*NCh2745:2013, NCh433.Of1996-Mod. 2009, NCh430.Of2008*
Material properties:	
Concrete:	*G25*
Reinforcing steel:	*A630-420H*
Material quantities:	
Designed by:	*Sergio Tito Cardozo Nava*
Sheet No.:	*3of 3*
Date:	*March 2025*

Structural element	Concrete (m^3)	Reinforcing steel (kg)	Reinforcement ratio (kg/m^3)
Columns	48.00	14620.00	304.58
Beams	121.59	15552.00	127.91
Slabs	170.47	15015.00	88.08
Total	**340.06**	**45187.00**	